VOLUME 85 NO. PO4
AUGUST 1959
PART 1

JOURNAL of the

Power

Division

PROCEEDINGS OF THE

AMERICAN SOCIETY
OF CIVIL ENGINEERS

BASIC REQUIREMENTS FOR MANUSCRIPTS

This Journal represents an effort by the Society to deliver information to the reader with the greatest possible speed. To this end the material herein has none of the usual editing required in more formal publications.

Original papers and discussions of current papers should be submitted to the Manager of Technical Publications, ASCE. Authors should indicate the technical division to which the paper should be referred. The final date on which a discussion should reach the Society is given as a footnote with each paper. Those who are planning to submit material will expedite the review and publication procedures by complying with the following basic requirements:

1. Titles should have a length not exceeding 50 characters and spaces.

2. A 50-word summary should accompany the paper.

3. The manuscript (a ribbon copy and two copies) should be double-spaced on one side of 8½-in. by 11-in. paper. Papers that were originally prepared for oral presentation must be rewritten into the third person before being submitted.

4. The author's full name, Society membership grade, and footnote reference stating present employment should appear on the first page of the paper.

5. Mathematics are reproduced directly from the copy that is submitted. Because of this, it is necessary that capital letters be drawn, in black ink, ⅛-in. high (with all other symbols and characters in the proportions dictated by standard drafting practice) and that no line of mathematics be longer than 6½-in. Ribbon copies of typed equations may be used but they will be proportionately smaller on the printed version.

6. Tables should be typed (ribbon copies) on one side of 8½-in. by 11-in. paper with a 6½-in. by 10½-in. invisible frame. Small tables should be grouped within this frame. Specific reference and explanation should be made in the text for each table.

7. Illustrations should be drawn in black ink on one side of 8½-in. by 11-in. paper within an invisible frame that measures 6½-in. by 10½-in.; the caption should also be included within the frame. Because illustrations will be reduced to 69% of the original size, the capital letters should be ⅛-in. high. Photographs should be submitted as glossy prints in a size that is less than 6½-in. by 10½-in. Explanations and descriptions should be made within the text for each illustration.

8. Papers should average about 12,000 words in length and should be no longer than 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

Further information concerning the preparation of technical papers is contained in the "Technical Publications Handbook" which can be obtained from the Society.

Reprints from this Journal may be made on condition that the full title of the paper, name of author, page reference (or paper number), and date of publication by the Society are given. The Society is not responsible for any statement made or opinion expressed in its publications.

This Journal is published bi-monthly by the American Society of Civil Engineers. Publication office is at 2500 South State Street, Ann Arbor, Michigan. Editorial and General Offices are at 33 West 39 Street, New York 18, New York. $4.00 of a member's dues are applied as a subscription to this Journal. Second-class postage paid at Ann Arbor, Michigan.

SM,PO,SA,

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

POWER DIVISION
EXECUTIVE COMMITTEE
John F. Bonner, Chairman; Marcel P. Aillery, Vice-Chairman;
George R. Strandberg; George J. Vencill; Richard R. Randolph, Jr., Secretary

COMMITTEE ON PUBLICATIONS
Thomas J. Cotter, Chairman; Alan J. Michael; James M. Mullarkey

CONTENTS

August, 1959

Papers

	Page
Bureau of Reclamation Practices for Control of Cracking in Arch Dams by Charles L. Townsend	1
The Thomas H. Allen Electric Generating Station by Peter J. McCoy and Vincent Shamamian	23
Multiple Purpose Power Plant Capacity by Richard E. Krueger	51
Ocean Cooling Water Systems for Two Thermal Plants by L. T. Mariner and W. A. Hunsucker	65
Discussion	87

Copyright 1959 by the American Society of Civil Engineers.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

U.S.B.R. PRACTICES FOR CONTROL OF CRACKING IN ARCH DAMS

Charles L. Townsend[1]

SYNOPSIS

This paper describes the temperature control studies undertaken by the Bureau of Reclamation and the temperature control measures most commonly used to prevent cracking in arch dams.

INTRODUCTION

The prevention of cracking in arch dams is of concern to the designer from the time the first preliminary layouts are on the drawing board until the full reservoir load is applied to the completed structure. Trial-load analyses are used to properly shape and proportion the dam to the site for the prescribed loading conditions. The shape and condition of the foundation, as established by foundation excavation during construction, may vary from that assumed in the design, and changes in section may be required to meet the actual conditions. During this design and construction period, the designs may be changed several times as trial-load analyses, temperature control studies, and actual construction conditions are adjusted to each other.

Temperature control studies, in addition to furnishing data required for trial-load analyses, are prepared to determine crack prevention measures to be employed, and the means by which a monolithic structure is to be obtained. Basically, to function in the manner designed, an arch dam must be a continuous structure from abutment to abutment when carrying its load. Special temperature control measures must be adopted, therefore, so that the contraction joints can be permanently closed by grouting before the reservoir loads are applied. Such measures include precooling of the concrete, artificial cooling of the concrete by means of embedded pipe systems, the use of a

Note: Discussion open until January 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2130 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. PO 4, August, 1959.

1. Engr., Concrete Dams Section, Div. of Design, Commissioner's Office, Bureau of Reclamation, Denver, Colo.

construction-slot, or any combination of these methods. The most commonly used of these measures for arch dams constructed by the Bureau of Reclamation is the pipe-cooling method, supplemented as necessary by the precooling method.

Early studies are based on existing data and on a possible construction schedule. Actual exposure conditions, water temperatures, and construction progress may vary widely from the conditions assumed, and adjustments are made during the construction period. Construction procedures and practices are anticipated during the design and specifications stage. Revisions and amendments are made during construction, when necessary, to obtain the best structure possible consistent with economy and construction problems.

Design Considerations

Trial Load Studies

Many factors influence the type of layout best suited for an arch dam at a particular site. The foremost of these factors are associated with topographic, geologic, and hydrologic conditions. From the standpoint of stress distribution, a symmetrical or nearly symmetrical profile is desirable, although not necessary. If the topography is such that a symmetrical profile does not exist, a layout suitable for a nonsymmetrical site may be developed by using fillets or by changes in the upstream and/or downstream radii, in such a manner as to equalize the stresses at the abutments of the dam. The width of the canyon has a major influence on the type of layout and on the stresses which will develop in the completed structure. The elastic properties of the foundation rock have an effect on the magnitude and extent of tensile areas in an arch dam, since load stresses are affected by the ratio of the modulus of the concrete to the modulus of the rock. An increase in this ratio of E_c/E_r tends to increase the tensile stresses occurring in the central portions of arches and on the downstream side of the cantilevers, but tends to reduce tensile stresses near the foundation extrados. Where the foundation rock has a relatively low strength, it may be necessary to develop a layout which will limit the stresses imposed on the rock to prescribed values. If large floods can be expected during the construction period, the means of diversion and care of the river during construction must be considered at the time the layout is being made, as the most economical structure may not be the most economical structure from the overall cost standpoint when the diversion and other construction difficulties are taken into consideration.

Objectionable tensile stresses will develop in arches of insufficient curvature. The largest central angle practicable, consistent with economy of construction, should therefore be used. In most layouts, it has been found that the largest practicable central angle for the top of the dam will vary from 90° to 110°.

An approximate thickness for the top arch may be found by the empirical formula,

$$T = 0.01\,(2R+H)$$

where R is the radius at the axis of the dam and H is the height of the dam at the maximum section. With the radius and thickness of the top arch established, a crown section and lines of centers are developed to give a trial layout for preliminary trial-load studies.

Preliminary trial-load studies are usually crown adjustments, a uniform loading being assumed across each arch element. In these crown adjustments, the external loads are divided between the crown cantilever and the arches in such a manner as to obtain equal radial deflections at the points of intersection. Several crown adjustment studies may be required to develop a layout which would be suitable for the selected site. Tensile stresses which could cause cracking of the structure are reduced to acceptable tensile stresses by modifications to the layout, or a new layout is made to obtain satisfactory stress conditions. Temperature loads, obtained from temperature control studies, are made a part of the analysis, and the benefit of a temperature rise is used to reduce tensions due to external loading. The distribution of loads in the structure can be changed by selecting different grouting temperatures in the dam. Construction of the dam in stages, with subsequent early storage of water, may be anticipated early in the design and taken into consideration in the trial-load studies.

In general, tensile stresses in arches tend to occur in the lower part of the dam, near the intrados of arches in the central part of the arch and near the extrados of arches near the abutments. Tensile stresses in cantilevers tend to occur on the upstream face of the cantilever near the base of the dam and on the downstream face of the cantilever near the top of the dam. The tensile stresses in the arches and the tensile stresses on the upstream face of the cantilevers can be minimized by keeping the arch radii as short as practicable, and by increasing the temperature benefit obtained by subcooling the concrete prior to grouting the contraction joints. The tensile stresses on the downstream face of the cantilever near the top of the dam can be reduced or eliminated by thinning the upper portion and by a section configuration which will give moments in the cantilever due to concrete weights in a downstream direction.

Temperature Control Studies

The measures required to obtain a monolithic arch structure and the measures necessary to reduce cracking tendencies to a minimum are determined by temperature control studies. Temperature control studies begin early in the design stage when programs are laid out to collect essential air and water temperature data. Topographic conditions, accessibility of the site, length of construction period, and the size and type of dam will have an effect upon, and will often influence, the size of blocks, rate of placement, and the temperature control and related crack prevention measures. The climatic conditions at the site, period of flood runoff, and reservoir operation requirements are studied to see what methods of temperature control are adaptable to the conditions existing during the construction period. The ever-growing collection of data and knowledge of the site, dam, and construction problems are gradually assembled so that decisions can be made on the nature and extent of the concrete temperature control, closure or grouting temperature, reduction of temperature gradients near exposed faces, insulation, and other crack prevention measures.

Mass concrete increases in temperature after placement due to the heat of hydration of the cementing materials. The maximum temperature is reached at age 2-1/2 to 4 days when cooling coils are embedded in the concrete, and at a somewhat later age when cooling coils are not used. The maximum temperature attained is primarily dependent on the placing temperature of the

concrete and the types and amounts of cement and pozzolan used in the mix. From the optimum temperature standpoint, this maximum temperature should not exceed the final stable temperature of the structure. This ideal condition will seldom, if ever, be encountered in the United States, in spite of the fact that concrete technology has advanced to the point where the strengths required for mass concrete can be obtained with a mix having an adiabatic temperature rise as low as 35° to 40° F. An approach to the ideal condition can be made, where the final stable temperature is relatively high, by precooling the concrete and placing at 45° to 50° F.

Numerous references are made, as above, to the final stable temperature of a massive concrete structure. Actually, such a condition does not exist as a single, definable temperature. A stabilized condition will eventually exist, however, in which the temperature at any given point in the structure will fluctuate between certain limits. These limits are related to the distance to each of the boundaries, the amplitudes and periods of the air and/or water temperatures at each face, the solar radiation received by each face due to its slope and orientation, and the diffusivity (conductivity divided by the product of specific heat times density) of the concrete. For those points near the downstream face of the dam, this fluctuation in temperature will reflect a daily air temperature cycle and solar radiation effect superimposed on an annual air temperature cycle. As the distance from the downstream face increases, the daily cycle will be damped out and the effect of the annual air temperature cycle, as modified by solar radiation, will become more apparent. Near the upstream face and below the minimum reservoir water surface, the concrete temperatures will reflect some part of an annual reservoir water temperature cycle. Fig. 1 shows the temperature variation with depth in concrete for a daily cycle and an annual cycle of exposure temperatures. The temperature distribution throughout a dam is therefore a complex and ever-changing condition. The temperature at any point can be computed theoretically by several methods. The validity of any such computation, however, is dependent upon the accuracy of the assumed exposure conditions.

Range of Mean Concrete Temperature

The temperature and temperature distributions which exist at any given time are of interest in several design problems, but cannot be used as such in the trial-load analysis to determine temperature stresses in arch dams. Of primary interest in the trial-load analysis is the range or amplitude of the mean concrete temperature for each of the arches used in the analysis. This mean concrete temperature of the arch will be sufficient to determine temperature stresses for most arch dams. In relatively short, thick arches, however, temperature gradients which exist between the upstream and downstream faces may also be taken into consideration.

The range of mean concrete temperature for each theoretical arch is obtained by applying estimated exposure conditions to the edges of a concrete slab of equal thickness. The exposure conditions are based on air and water temperature records at the site, if available, or on data obtained from nearby stations and adjusted to the site. The reservoir water temperatures are estimated by considering the reservoir operation, source of water, ratio of annual river runoff to reservoir capacity, climatic conditions between the source and the reservoir, and climatic conditions at the reservoir site. Reservoir temperatures at given depths and at different times of the year have been

obtained on a number of dams constructed by the Bureau of Reclamation, and these data aid in estimating temperatures at other sites. The effects of solar radiation are obtained from the slope and orientation of the concrete surfaces and the latitude and topography of the site.(1) Thermal properties of the concrete are estimated after probable aggregate sources are determined,(2) or are determined by laboratory tests.

Normally, only the condition of reservoir full is considered in determining the range of mean concrete temperature. When stage construction is taken into consideration or where extremes of weather occur with partial drawdown, further studies are prepared to determine concrete temperatures existing at given times of the year.

Figs. 2 and 3 show the general features and temperature conditions for Hungry Horse Dam in Montana, and Figs. 4 and 5 show similar data for Monticello Dam in California. Both dams were designed and constructed by the Bureau of Reclamation.

Closure Temperature

After the ranges of mean concrete temperature are determined, the closure or grouting temperature is selected. The closure temperature is normally based upon what the trial-load analyses show to be desirable from a stress standpoint, but may be determined by practical or economic considerations.

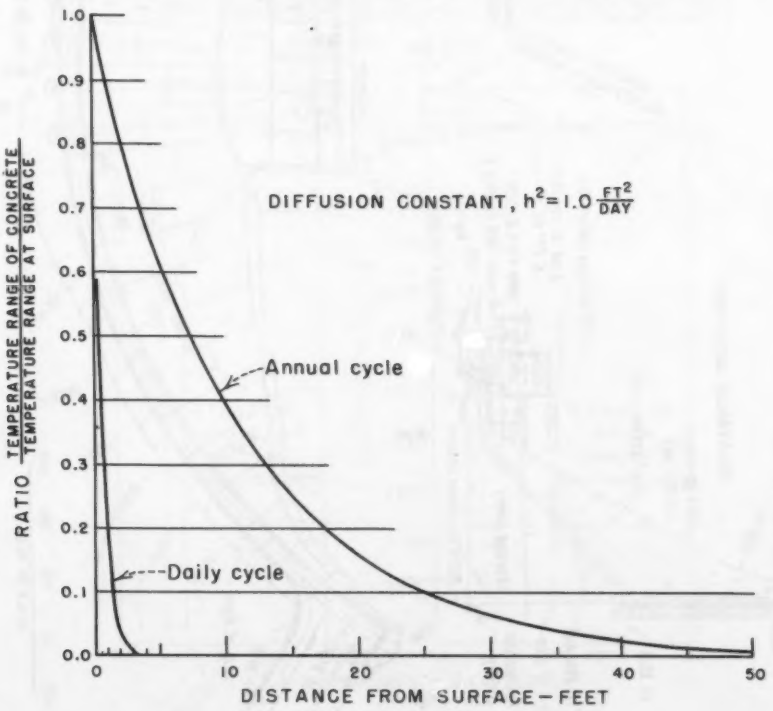


Fig. 1

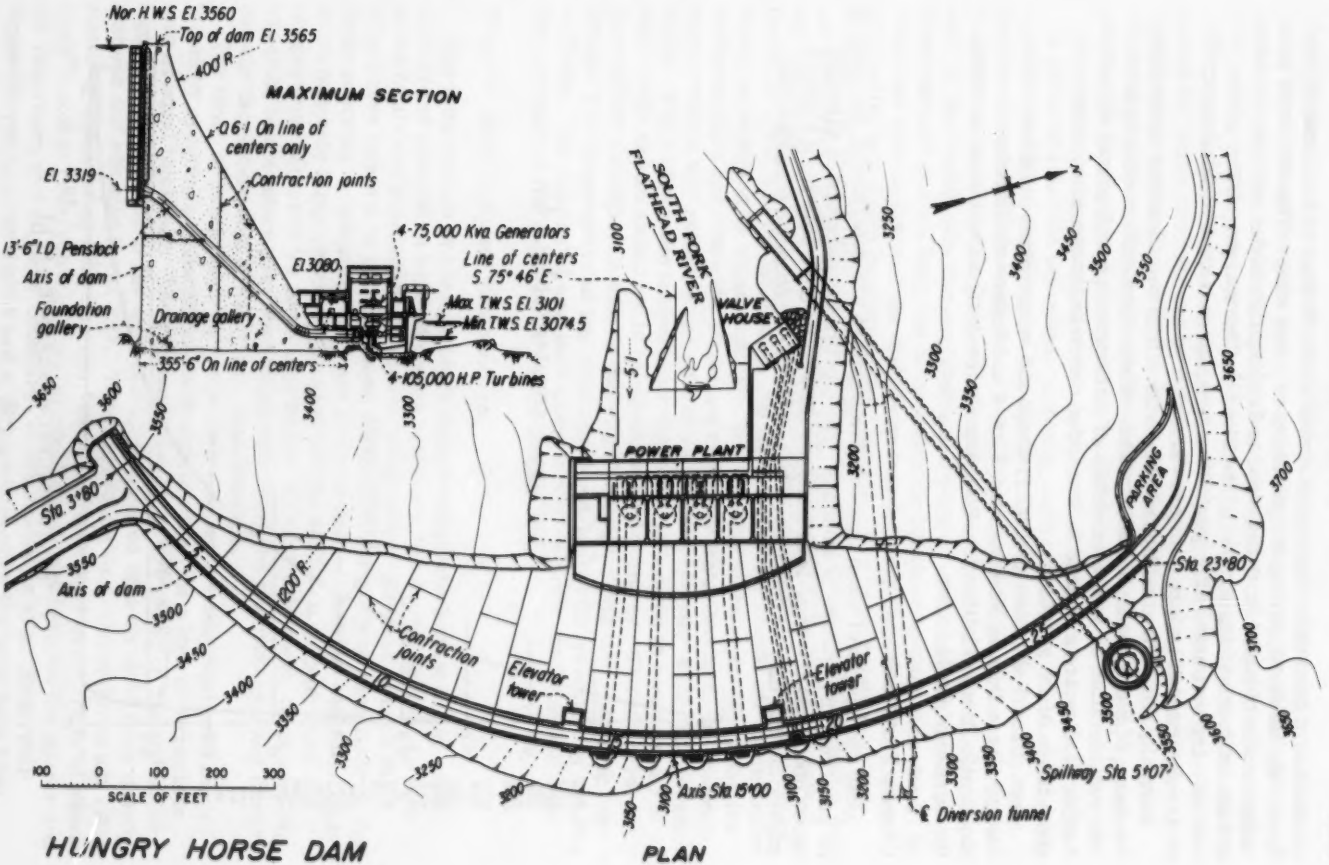
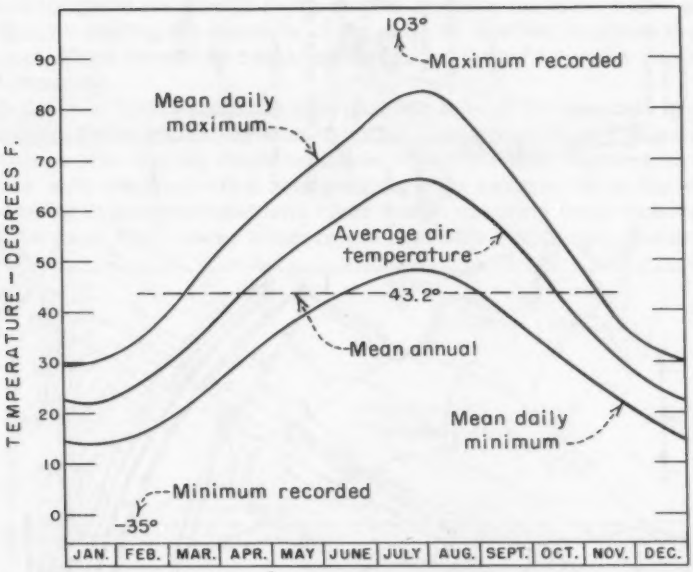

Nor. H.W.S. El 3560
Top of dam El 3565
400' R
MAXIMUM SECTION
0.6:1 On line of centers only
El 3319
Contraction joints
13'-6" I.D. Penstock
Axis of dam
Foundation gallery
Drainage gallery
El 3080
4-75,000 Kva Generators
Line of centers S 75° 46' E
Max. T.W.S. El 3101
Min. T.W.S. El 3074.5
355'-6" On line of centers
4-105,000 H.P. Turbines
SOUTH FORK FLATHEAD RIVER
VALVE HOUSE
POWER PLANT
PARKING AREA
Sta 3+80
Sta 23+80
Axis of dam
200' R
Contraction joints
Elevator tower
Elevator tower
Axis Sta 15+00
℄ Diversion tunnel
Spillway Sta 5+07
100 0 100 200 300
SCALE OF FEET
HUNGRY HORSE DAM
PLAN

Fig. 2



HUNGRY HORSE DAM

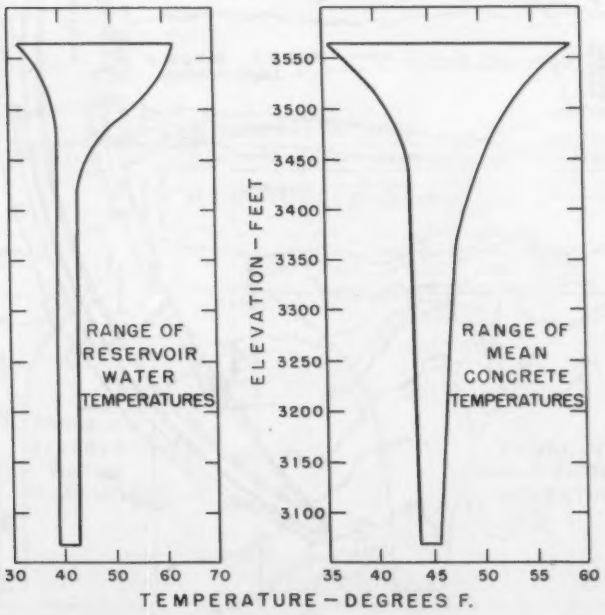
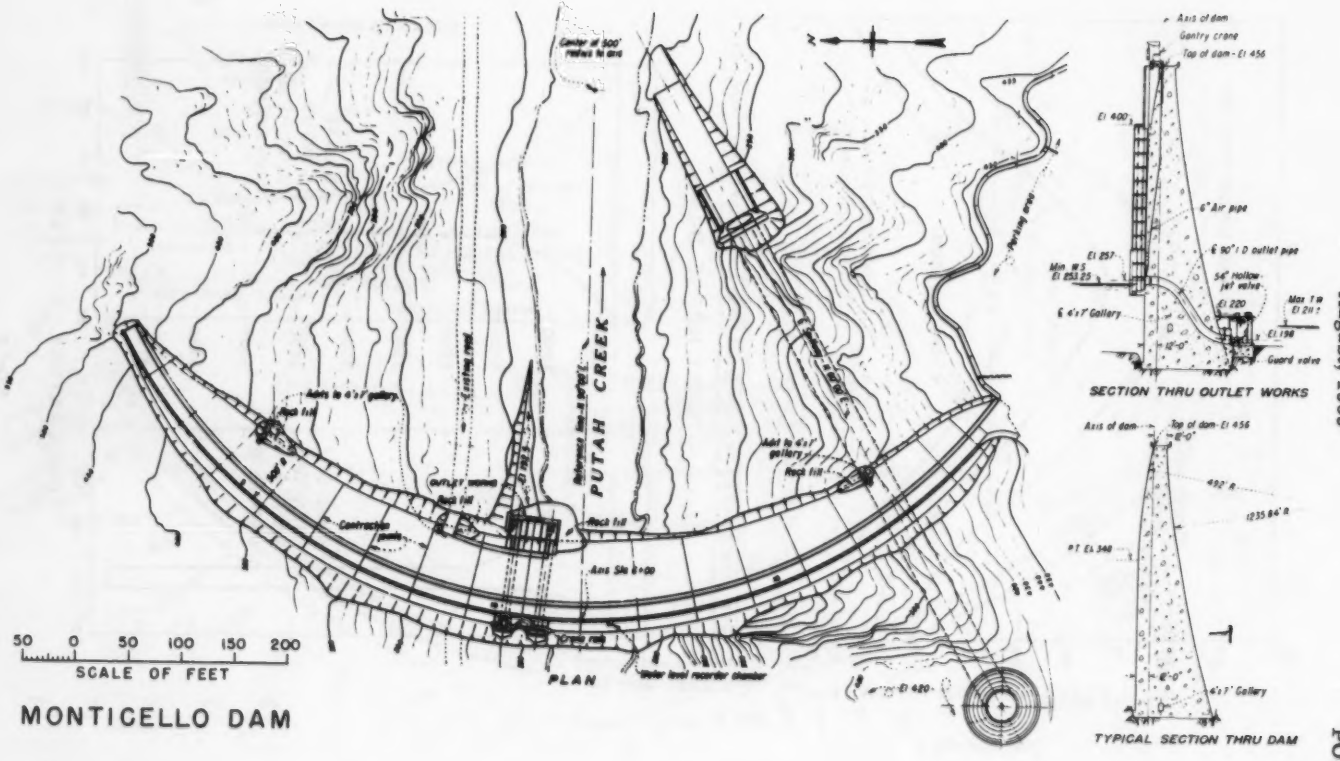


Fig. 3

HUNGRY HORSE DAM PLAN Fig. 2

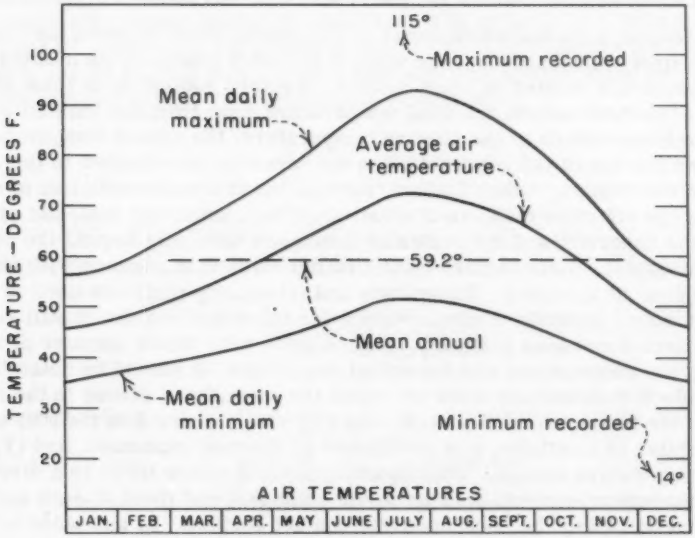


MONTICELLO DAM

Fig. 4

The designer quite often has to make a design decision whether to lose 2 to 5 degrees of temperature benefit in the arches by using the river water available for artificially cooling the concrete of the dam, or whether to obtain the desired temperature benefit by requiring mechanically refrigerated water to cool the concrete.

From the practical standpoint, it is possible to cool the concrete by means of an embedded pipe cooling system to within 4 or 5 degrees of the mean temperature of the cooling water available. Concrete temperatures as low as 35° F have been obtained with a refrigerating plant using brine as the coolant. Where cooling is accomplished with river water, concrete temperatures depend on the mean river water temperature available. At Hungry Horse Dam,



MONTICELLO DAM

Fig. 4

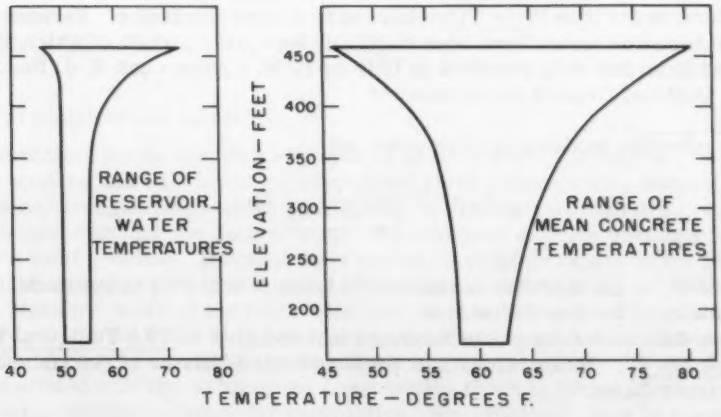


Fig. 5

32° to 34° F river water was available during the colder months of the year and pipe cooling was accomplished exclusively with this river water. A single closure temperature of 38° F was determined satisfactory for the particular layout and section of Hungry Horse Dam. At Monticello Dam, river water was both limited in quantity and was relatively warm since the stream primarily carries a surface runoff during periods of rainfall. Refrigeration of the water was therefore required to obtain the desired closure temperatures. Two closure temperatures were used at Monticello Dam, 45° F in the lower part of the dam and 55° F in the upper part of the dam, so that more load could be carried by the lower part of the dam.

Temperature Control Versus Length of Construction Block

The tensile stresses which occur in a concrete block between the time the block is first placed and the time when it has been cooled to its minimum temperature are related to many factors. Factors subject to at least some degree of control include the total temperature drop from the maximum concrete temperature to the closure temperature, the rate of temperature drop, and the age of the concrete when the concrete is subjected to the temperature change. Other factors, such as the thermal coefficient of expansion, the effective modulus of elasticity,* the elastic and inelastic properties of the concrete, and the restraint factor are normally beyond the control of the designer. These factors cannot be determined in advance with any great degree of accuracy. Experience and laboratory tests are used to the greatest extent possible to obtain values for these factors, but it still remains that the actual stresses will vary between quite wide limits because of variances in the assumptions and the actual conditions. It should be noted that the size of block theoretically does not affect the intensity of stress in the normal equation for temperature stress, $S = Ee\ (T_2 - T_1)$, where S is the unit stress, E is modulus of elasticity, e is coefficient of thermal expansion, and $(T_2 - T_1)$ is the temperature change. This equation is valid where there is a direct restraint against movement, such as would exist in a rod fixed at each end, or such as exists along the rock-concrete contact surface. At any point off the foundation surface, however, only an indirect restraint exists, the value of which is a function of the length of the block and the height above the foundation. The equation for temperature stress in a large concrete block should therefore be $S = REe\ (T_2 - T_1)$, where R is a restraint factor. Various values of this restraint factor have been obtained, the most common of which are derived from test data reported in 1940 by R. W. Carlson and T. J. Reading to the Portland Cement Association.**

*The effective modulus is often given as:

$$E_{eff} = \frac{1}{1 + \frac{0.4\ E_c}{E_f}}\ E_c,$$

where E_c is the modulus of elasticity of concrete and E_f is the modulus of elasticity of the foundation rock.

**These data have been put in diagrams and included in TVA Technical Monograph No. 67, "Measurements of the Structural Behavior of Norris and Hiwassee Dams."

Aside from the degree of restraint, cracks may occur because the larger blocks cover a greater area of the foundation and are thereby subjected to a greater number of stress concentrations due to the physical irregularities and variable composition of the foundation. Too, cracks may occur because of the construction schedule and construction operations. Longer blocks are more likely to have cold joints occurring during placement of the concrete, and these cold joints form definite planes of weakness. The longer blocks may also be exposed for longer periods of time, allowing extreme temperature gradients to form near the surfaces. The stresses caused by these steep temperature gradients will then cause cracks to form along any planes of weakness which may then exist.

Because the stress-producing factors are either unknown, varying with age, or susceptible to change during construction, it is difficult to arrive at a definite degree of control for any given size of block. For concrete arch dams, therefore, and as a guide only, Table I shows typical temperature control measures adopted during the early stages of design.

Table I

Block length	Treatment		
Over 240 feet	Use longitudinal joint. Maximum length between longitudinal joints to be 180 feet.		
	Fdn to H = 0.2 L*	H = 0.2 L to 0.5 L*	Over H = 0.5 L*
	Hold temperature drop (from maximum concrete temperature to grouting temperature) to a maximum of		
180 feet to 240 feet	30 degrees	35 degrees	40 degrees
120 feet to 180 feet	35 degrees	40 degrees	45 degrees
90 feet to 120 feet	40 degrees	45 degrees	No restriction
60 feet to 90 feet	45 degrees	No restriction	No restriction
Up to 60 feet	50 degrees	No restriction	No restriction

*Height above foundation = H; block length = L.

Width of Construction Block

Contraction joints are normally spaced 50 feet apart, but may be controlled by the spacing and location of penstocks and river outlets, or by definite breaks and irregularities of the foundation. A spacing as uniform as possible is maintained so that the contraction joints will have uniform openings for contraction joint grouting. Spacings have varied in dams designed by the Bureau of Reclamation from 30 feet to about 80 feet as measured along the axis of the dam. Since the width of the block controls the size of construction plant, an inducement can be made to the smaller contractors by having block widths of 30 to 50 feet in the smaller concrete dams.

Associated with the width of the construction block is the problem of obtaining good groutable openings for contraction joint grouting. This is especially

true when the blocks are approximately 30 feet in width. In this case it may become necessary to cause a larger temperature drop to take place to open the joint than would otherwise occur. Such a temperature drop is not without limit, because the long dimension of the block may still require the temperature drop to be held to a minimum.

Control of Temperature Drop

Control of the temperature drop from the maximum concrete temperature to the grouting temperature is the one factor relating to stress development over which a measure of control can be achieved. A reduction in the total temperature drop may be accomplished by reducing the temperature rise which occurs immediately after placement of concrete, by reducing the placing temperature of the concrete, or by adopting both measures.

The temperature rise is normally controlled by the selection of the type and amount of cement used in the concrete, the replacement of part of the cement by a pozzolan, and the artificial cooling of the concrete through an embedded pipe system during the first few days after placement. Minor benefits may also accrue due to evaporative cooling when the concrete is water cured and due to a slower rate of hydration when placed at lower temperatures. In most instances, Type II cement will produce concrete temperatures which are acceptable. In the smaller structures, Type I cement will usually be satisfactory. Type II cement can be obtained, in many cases, at little or no increase in cost over the more common Type I cement. If this is the case, Type II is selected because of its better resistance to sulfate attack, better workability, and lower permeability. Type IV cement, although originally developed for use in mass concrete to reduce the temperature rise, is now used only when other methods of control will not accomplish the desired result. For example, it may be used near the base of long blocks where a high degree of restraint exists. Concrete with Type IV cement requires more curing than concrete with other types of cement, and extra care is required at early ages to prevent damage to the concrete from freezing or from too early a form removal.

Concrete technology has reached the point where the old standard of four sacks of cement per cubic yard for mass concrete is no longer the criterion. Present-day practice is to use about three sacks of cementing materials per cubic yard of concrete for the interior mass of the dam. The cementing materials may be entirely cement or may be a mixture of cement and a pozzolanic material. Since the temperature rise in concrete is directly related to the amount of cement and to the amount of pozzolan, and since the heat generation of the pozzolanic replacement is roughly 50 per cent that of cement, the subsequent temperature rise in present-day concrete has been reduced by as much as 40 per cent over that obtained in the past. Fig. 6 shows typical temperature rise curves for concrete placed in Monticello and Hungry Horse Dams.

Reducing the placing temperature of the concrete will result in approximately an equal reduction in the maximum concrete temperature. Various methods have been used to reduce concrete placing temperatures, and the method or combination of methods will vary with the degree of cooling required and the construction contractor's previous experience. Cooling of the coarse aggregate has varied from shading and sprinkling the aggregate in the stockpiles to chilling the aggregate in large tanks where the aggregate is

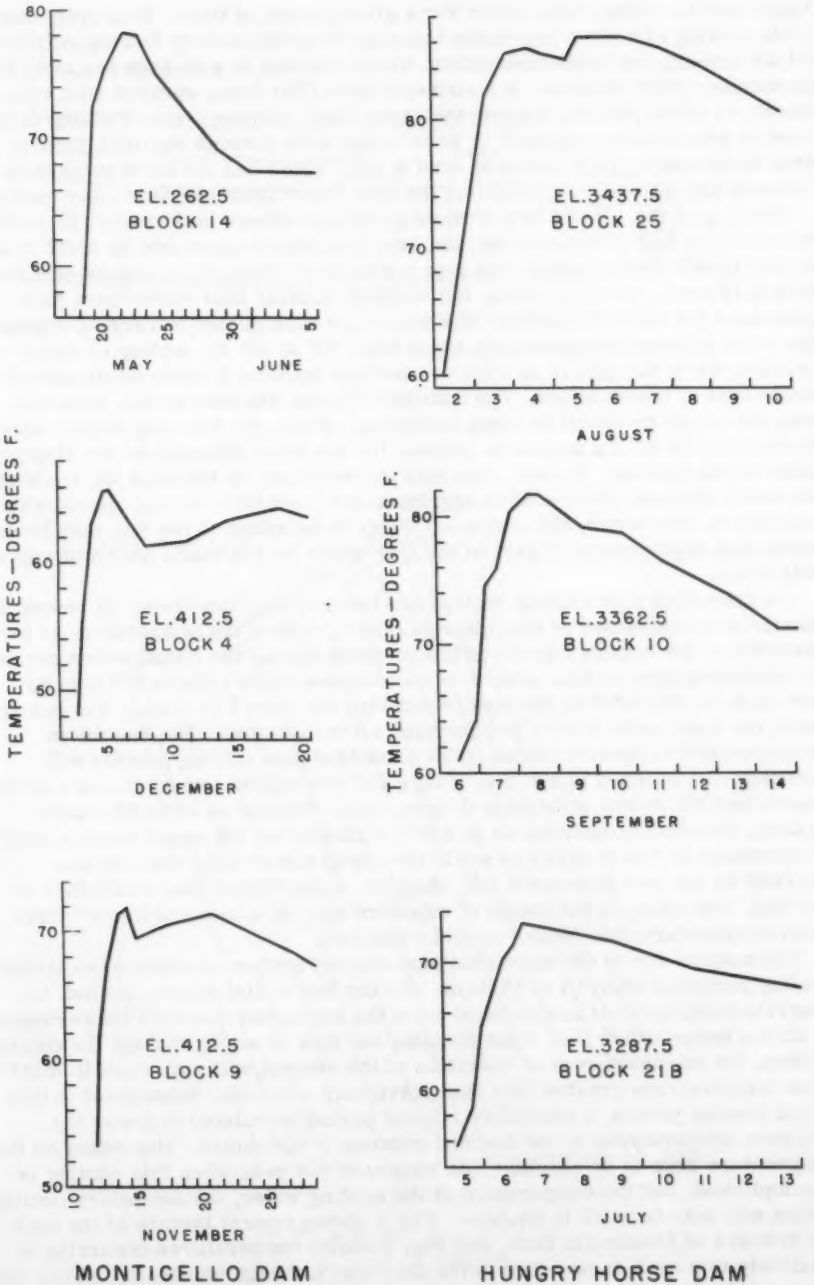

EL.262.5
BLOCK 14
MAY
JUNE
EL.3437.5
BLOCK 25
AUGUST
TEMPERATURES—DEGREES F.
EL.412.5
BLOCK 4
DECEMBER
EL.3362.5
BLOCK 10
SEPTEMBER
EL.412.5
BLOCK 9
NOVEMBER
MONTICELLO DAM
EL.3287.5
BLOCK 21B
JULY
HUNGRY HORSE DAM

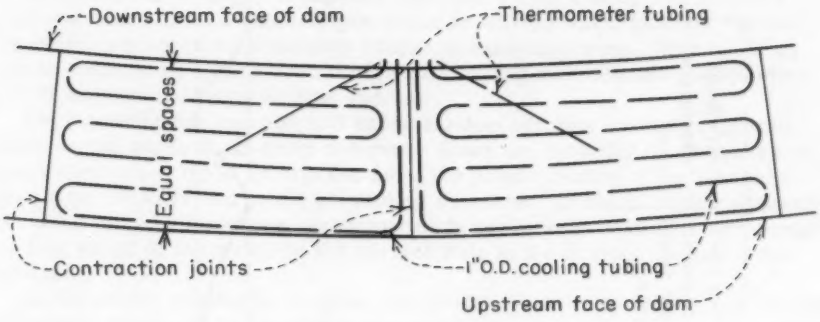
Fig. 6

immersed in refrigerated water for a given period of time. Relatively complete cooling of coarse aggregate has also been obtained by forcing refrigerated air through the aggregates either while draining in a storage bin after being immersed, while draining on a conveyor belt after being sprayed with cold water, or while passing through the bins of the batching plant. Cooling of the sand is usually accomplished by passing the sand through vertical, tubular heat exchangers. Immersion of sand in cold water has not been successful because of the difficulty in removing the free water from the sand after cooling.

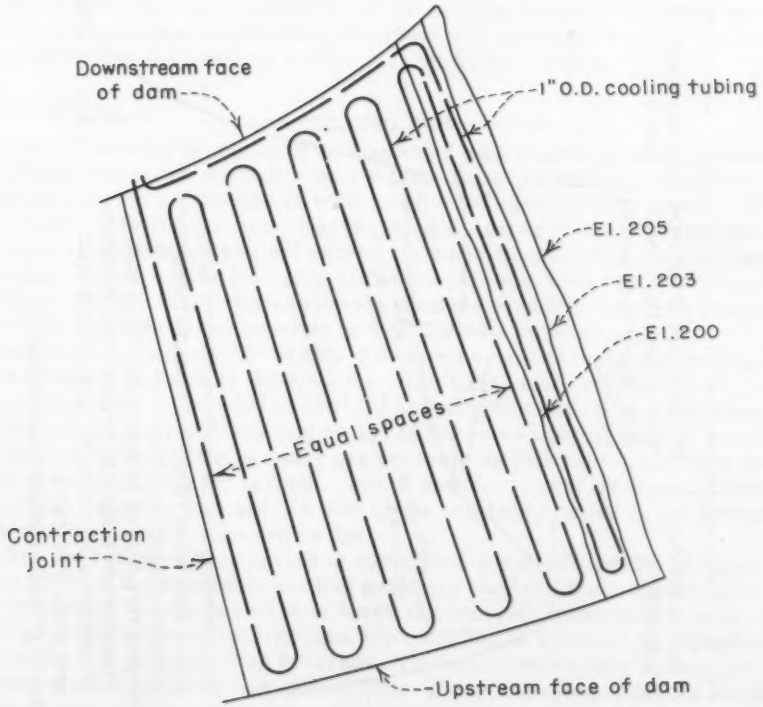
Cooling of the cement is a definite problem. Where bulk cement is obtained at relatively high temperatures, the best that can be hoped for, in some cases, is that it will cool naturally and lose a sizable portion of the excess heat before it is used. In other cases, the vertical, tubular heat exchangers have been used for cement cooling. Mix water has been cooled to varying degrees, the more common temperatures being from 32° to 40° F. Adding slush or crushed ice to the mix is an effective method because it takes advantage of the latent heat of fusion of ice. The addition of large amounts of ice, however, may not be too practical in some instances. First, the batching hopper must be designed in such a way as to prevent the ice from adhering to the sloping sides of the batcher. In part, this may be overcome by batching the ice with the sand. Second, if the coarse aggregate and sand both contain appreciable amounts of free water, the additional water to be added to the mix may be so small that replacement of part of the free water by ice would not be of appreciable value.

An embedded pipe cooling system has two primary functions: to reduce the maximum temperature of the concrete, and to reduce the temperature of the concrete to the required grouting temperature during the construction period. An embedded pipe cooling system cannot economically remove the heat as fast as it is generated in the concrete during the first 2 to 4 days, but can remove the heat faster than it is generated after that time. The maximum temperature of concrete cooled by an embedded pipe cooling system will, therefore, be reached at age 2 to 4 days and subsequent temperatures can be maintained below that maximum temperature. Without an embedded pipe system, concrete temperatures in a 5-foot placement lift would reach a peak temperature at 3 to 5 days and would then drop slowly until that lift was covered by the next placement lift. Another temperature rise would then occur and, depending on the length of exposure period, a new and higher maximum temperature than before could be obtained.

The normal use of the embedded pipe cooling system consists of an initial cooling period of from 10 to 15 days. During this initial cooling period, the concrete temperatures are reduced from the maximum concrete temperature to such a temperature that, upon stopping the flow of water through the cooling system, the continued heat of hydration of the cement will not result in maximum temperatures greater than that previously obtained. Subsequent to this initial cooling period, a secondary cooling period is utilized to lower the concrete temperatures to the desired grouting temperature. Depending on the temperature drop to be obtained, the season of the year when this cooling is accomplished, and the temperature of the cooling water, the secondary cooling period will take from 30 to 60 days. Fig. 7 shows typical layouts of the cooling systems at Monticello Dam, and Fig. 8 shows temperatures occurring in relatively thin arch dams. The latter diagram, although schematic, shows the ranges of times and temperatures experienced on dams constructed since 1949 by the Bureau of Reclamation.



PLAN AT EL. 400
BLOCKS 11 AND 12 SHOWN.



PLAN AT EL. 200
BLOCK 14 SHOWN.

Fig. 7

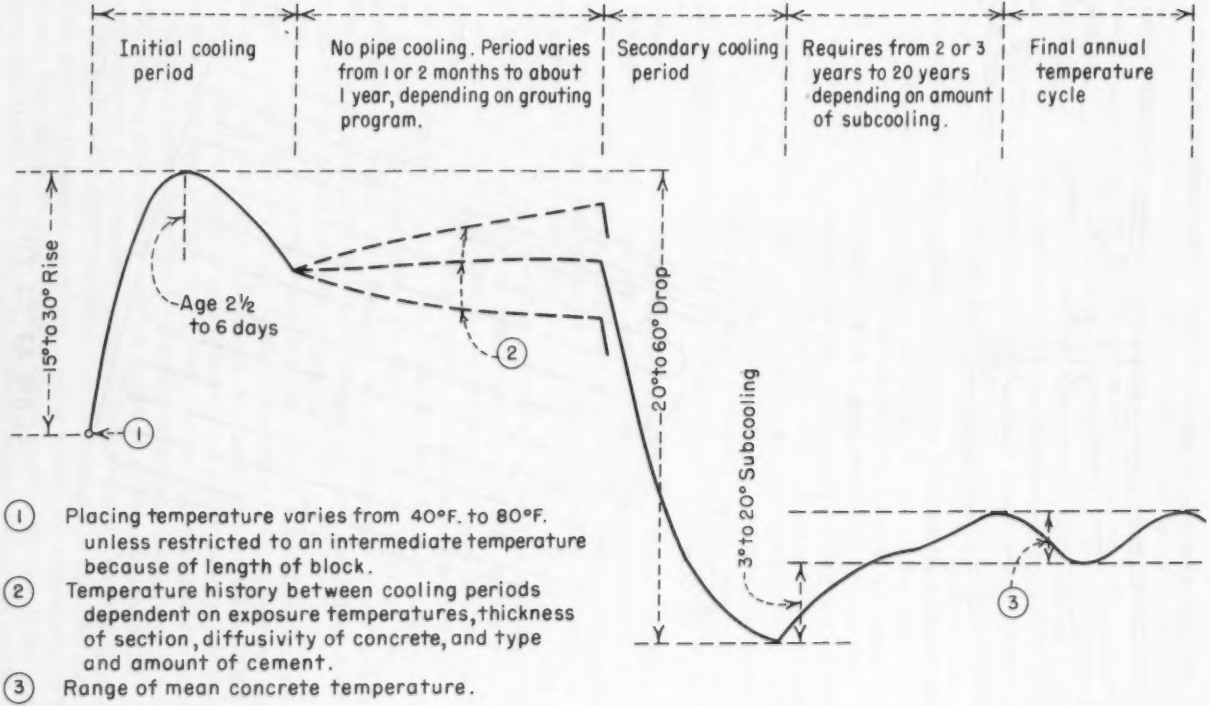

Initial cooling period
No pipe cooling. Period varies from 1 or 2 months to about 1 year, depending on grouting program.
Secondary cooling period
Requires from 2 or 3 years to 20 years depending on amount of subcooling.
Final annual temperature cycle
15° to 30° Rise
Age 2½ to 6 days
20° to 60° Drop
3° to 20° Subcooling
1
2
3
1 Placing temperature varies from 40°F. to 80°F. unless restricted to an intermediate temperature because of length of block.
2 Temperature history between cooling periods dependent on exposure temperatures, thickness of section, diffusivity of concrete, and type and amount of cement.
3 Range of mean concrete temperature.

Fig. 8

Surface Gradients

The cracking tendency at exposed surfaces where a steep temperature gradient exists from a cold surface to the relatively warm interior is often the starting point of objectionable cracks in mass concrete. This tendency can be reduced by a combination of proper curing methods and the effective use of an embedded pipe cooling system.

Drying shrinkage can and will cause random hairline cracks of limited depth on the surfaces of mass concrete. Since the majority of all cracks in mass concrete start at an exposed face, the proper curing of the surfaces should be considered a crack prevention measure. Although curing compounds are used in some instances, water curing is preferred because the evaporative cooling effect of the water on the surface aids in the overall cooling of the concrete.

In the colder months of the year, the tendency of the surface of the concrete is to drop rapidly to the exposure temperature. At the same time, the interior of the mass may still be increasing in temperature. Extreme temperature gradients are thus formed near the surface which, because of the restraint of the interior on the exterior, can cause severe cracking. An embedded pipe cooling system will cause a lowering of the interior temperatures and, although not undergoing as fast or as complete a temperature drop as the surface, will reduce the tendency to form cracks at the surface.

Rate of Cooling

Artificial cooling by means of an embedded pipe system is accomplished by circulating cold water through 1-inch outside diameter pipe or tubing laid grid-like over the top surface of each 5- or 7-1/2-foot lift of concrete. The rate of cooling is controlled so that the tensions set up in the concrete due to the drop in temperature will not exceed the tensile strength of the concrete. Continued cooling during the early age of the concrete with a rate of temperature drop of about 1° per day can create tensions which are equal to or exceed the tensile strength of the concrete in 3 to 4 weeks while undergoing a temperature drop of only 26° to 28°. An extreme rate of temperature drop can cause cracking at about 11 days, on the other hand, while undergoing a temperature drop of only 20° to 22°. To prevent the artificial cooling from causing cracks, an initial cooling period of not more than 2 weeks is usually specified, and the cooling systems are operated so that the temperature drop is not more than 1 degree per day. Fig. 9 shows a typical increase in tensile strength of concrete with age, as well as the tensions created in the concrete for different rates of temperature drop.

After the initial cooling period is completed, a variable period of time elapses before the secondary cooling period is started. Since the purpose of this secondary cooling period is to lower the concrete temperature to the desired grouting temperature, the time when cooling is resumed is dependent on the contraction joint grouting program. The temperature drop during the secondary cooling period has caused little, if any, cracking of blocks for the range of temperatures and temperature drops experienced by the Bureau of Reclamation.

The rate of temperature drop forced upon the concrete by an embedded pipe cooling system can be controlled by varying the length of coil, the horizontal spacing of the tubing, and velocity of the cooling water, and the temperature of the cooling water in the cooling systems. The length of coil and the

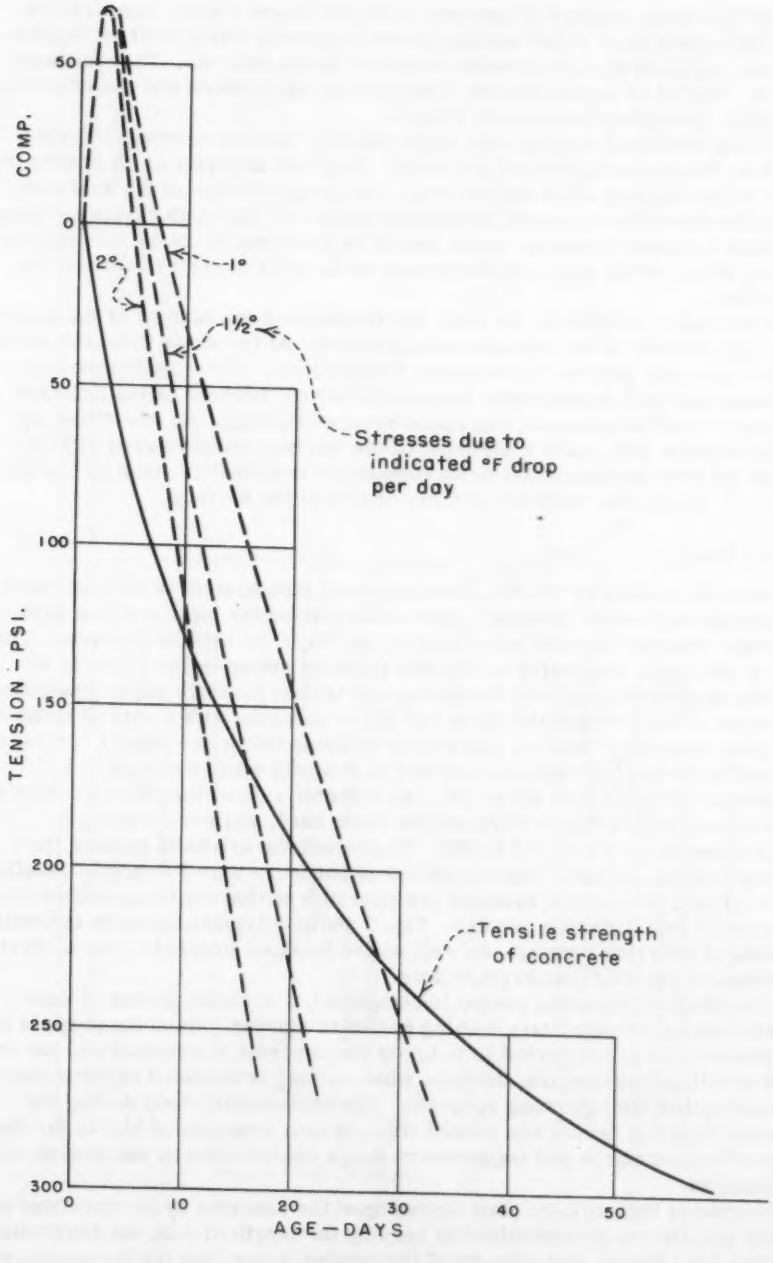

COMP.
50
0
50
100
150
200
250
300
TENSION – PSI.
2°
1°
1½°
Stresses due to indicated °F drop per day
Tensile strength of concrete
0
10
20
30
40
50
AGE – DAYS

Fig. 9

horizontal spacing are determined by the size of the blocks and the purpose of the cooling systems; that is, whether the cooling systems are primarily for lowering the temperature of the concrete prior to contraction joint grouting, for controlling the maximum concrete temperature, for reducing temperature gradients near exposed surfaces, or any combination of these. The effects of changes during the construction period such as the type or amount of cement used in the concrete, curing methods employed, exposure temperatures varying from those assumed, or any other factors which influence concrete temperatures, are normally taken care of by varying the periods of flow and the temperature of the cooling water. If these changes become major changes and vary greatly from those assumed, the temperature effects can be taken care of by varying the horizontal spacing of the tubing and/or length of cooling coil in those portions of the dam then being constructed.

Construction Requirements

Foundation Irregularities

Although the trial-load analyses assume relatively uniform foundation and abutment excavations and the design drawings show the desired excavation for the dam, the final excavation may vary widely from that assumed. Faults or crush zones are often uncovered during excavation, and the excavation of the unsound rock leaves definite irregularities. Where these areas are extensive, backfill concrete is placed and cooled before additional concrete is placed over the area. Unless the backfill concrete has undergone most of its volumetric shrinkage at the time the overlying concrete is placed, cracks can occur in the overlying concrete near the boundaries of the backfill concrete as loss of support occurs within the area of backfill concrete. Similar conditions exist where the foundation undergoes abrupt changes in slope. At the break of the slope, cracks often occur because of the differential movement which takes place between concrete held in place by rock and concrete held in place by previously placed concrete which has not undergone its volumetric temperature shrinkage. In both conditions, artificial cooling systems in conjunction with delays in placing at that location can be employed to obtain the volume change in the concrete below before proceeding with construction.

Relaxation of Initial Cooling

In the early spring and late fall months when exposure conditions can be severe, the length of the initial cooling period and the rate of temperature drop can be critical. This is especially true with relatively thin sections where pipe cooling combined with low exposure temperatures can cause the concrete temperature to drop too fast. In this case, artificial cooling should be stopped after 4 to 5 days and the concrete should be allowed to cool in a natural manner. In structures with thicker sections, the exposure temperatures have less effect on the immediate temperature drop, and the initial cooling period can be continued with the primary purpose of controlling the temperature difference between the exposed faces and the interior. A Schmidt's method of computation,* using estimated exposure conditions, will

*The method developed by E. Schmidt was published in "A. Foeppl's Festschrift," 1934, and is described in "Resistance to Cracking of Surface Layer of Concrete Gravity Dams," by Nielander, Second Congress on Large Dams, 1936.

usually suffice to determine the conditions existing for intermediate sections.

Insulation

Insulation is employed primarily to prevent steep temperature gradients near the exposed surfaces of the blocks. This may be accomplished by simply leaving wood forms in place, or may require the use of commercial-type insulation applied to the faces of the exposed block. Tops of blocks have been protected with sand or saw-dust where an extended exposure period was anticipated. Whatever the type of insulation, measures should be taken to exclude as much water or moisture from the insulation as practicable. For a short period of exposure, small space heaters may be used, either by themselves or in conjunction with canvas coverings which enclose the work.

Height Differential

A maximum height differential between adjacent blocks is specified in construction specifications for an arch dam for two purposes. First, from a temperature standpoint, an even temperature distribution throughout the structure will be obtained when all blocks in the dam are placed following a uniform and continuous placement schedule. This even temperature distribution is desirable, not only because of the subsequent uniform system of contraction joint openings, but also because the overall stress distribution will be nearer that assumed in the design. Extreme temperature gradients on the exposed sides of blocks can also be the start of circumferential cracks across the blocks. The chances of such cracks forming will be lessened when each lift is exposed for a minimum length of time. The second reason for the height differential is that it will cause construction of the dam to progress uniformly up from the bottom of the canyon. Contraction joints can then be grouted in advance of a rising reservoir, thus permitting storage at earlier times than would occur if construction progress was concentrated in selected sections of the dam. An additional maximum height differential, that occurring between the highest and lowest blocks in the dam, should be required if the early storage is desirable.

Where one or two blocks are left low for diversion or as a construction expedient, several adverse conditions may occur as construction progresses. These conditions arise when a large height differential exists during a long period of cold weather. Under such a situation, a deflection or tilting of the high block out into the area of the low block occurs because of the temperature differential between the cold face adjacent to the low block and the far face which remains relatively warm. This deflection can damage the metal seals installed for contraction joint grouting on the far face of the block if the deflection becomes too great. The deflection may also prevent continued placement of concrete in the block adjoining the far face of the high block. Continued placement in that block while the high block is deflected out into the opening of the low block will progressively wedge the high block out. During the following warm season, when the cold exposed face warms up, the high block will tend to return to its original position, causing a shearing movement upward with the possibility of horizontal cracks being formed in the adjoining block. These problems peculiar to a high height differential can be met and corrected by the timely use of the embedded pipe cooling systems.

The actual height differential is a compromise between arriving at the uniform temperature conditions and construction progress desired, and the

contractor's placement problem. In general, the maximum differential between adjacent blocks is stipulated as 25 feet where 5-foot lifts are used or 30 feet where 7-1/2-foot lifts are used. The maximum differential between the highest block in the dam and the lowest blocks in the dam is usually limited to 40 feet where 5-foot lifts are used and to 52.5 feet where 7-1/2-foot lifts are used.

Openings in Dam

Where possible, galleries and large openings in the dam should be located in regions of low stress or where cracks would not be detrimental to structural stability. Because openings concentrate stresses at their edges, all possible means should be used to prevent the beginning of cracks from the surfaces of such openings. All galleries and large openings are reinforced for internal stresses and temperature variations. Proper curing methods should be used at all times. The entrances to such openings should be bulkheaded and kept closed, with self-closing doors where traffic demands, to prevent the circulation of air currents through the openings. Such air currents not only tend to dry out the surfaces but can cause extreme temperature gradients to form.

Extended Exposure of Horizontal Construction Joints

Horizontal construction joints occurring at the tops of lifts left exposed over a winter's season are particularly vulnerable to surface tensions during the annual temperature cycles which occur during the life of a structure. These construction joints are cold joints in every sense of the word, and it is difficult to obtain an effective bond between the old concrete and the new concrete. Horizontal cracks often occur at the location of these cold joints during subsequent winter exposures, surface tensions causing the crack to extend into the concrete along the construction joint plane as much as 10 or 12 feet.

Preventive measures used are those directed toward maintaining the bond between the old and new concrete. In addition to preparing the surface physically for placement of fresh concrete, considerable benefit is obtained by warming the top 15 to 20 feet of the previously placed concrete by circulating warm water through the embedded cooling coils before new concrete is placed. In conjunction with warming the old concrete, the horizontal spacing of the cooling coils in the first 2 or 3 lifts of new concrete is reduced to hold down the temperature rise. These measures will create a temperature differential in the concrete over a height of 25 to 35 feet which will reduce the tendency to shear or break bond on the horizontal construction joint plane during subsequent temperature drops.

REFERENCES

1. "The Average Temperature Rise of the Surface of a Concrete Dam Due to Solar Radiation," by W. A. Trimble, Bureau of Reclamation memorandum, August 27, 1954.

2. "Thermal Properties of Concrete," Bulletin 1, Part VII, Boulder Canyon Project Final Reports (Table 8).

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

THE THOS. H. ALLEN ELECTRIC GENERATING STATION

Peter J. McCoy[1] and Vincent Shamamian,[2] M. ASCE

ABSTRACT

This paper reports the civil engineering features and the solution of the related engineering problems resulting from the natural site conditions encountered as well as from those features developed to provide for the operating and building requirements of a modern plant of this capacity.

SYNOPSIS

Since 1938, the City of Memphis, Tennessee, has purchased its electrical power from the Tennessee Valley Authority. In 1955, when the City of Memphis undertook to install its own generating facilities, a complete and detailed program of engineering and construction was developed to meet the projected 1958-'59 power requirements and to continue meeting the expected demands of the years to follow. The first step in this program is now nearing completion. This is the construction of the initial three units of the Thos. H. Allen Electric Generating Station.

The successful accomplishment of this program within the time scheduled placed a heavy burden on the engineers. To the Civil Engineer fell a large part of the responsibility for evaluating and selecting the most suitable and economical site for the plant. Many important engineering decisions had to be made in rapid sequence in order to assure completion of the first phase of the City's program within the span of time allotted for it.

The purpose of this paper is to report the civil engineering features resulting from the natural site conditions encountered as well as from those features developed to provide for the operating and building requirements of a modern

Note: Discussion open until January 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2131 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. PO 4, August, 1959.

1. Chf. Civ. Engr., Burns and Roe, Inc., New York, N. Y.
2. Superv. Civ. Engr., Burns and Roe, Inc., New York, N. Y.

plant of this capacity. A description of these features would be incomplete without describing some of the many engineering problems that had to be solved.

INTRODUCTION

The City of Memphis, on the east bank of the Mississippi River, has been undergoing an unusual growth in commerce and industry. Naturally, this growth has resulted in a rapid increase in population. The resulting demand for electrical power, measured by the annual maximum-hour peak demands, has reflected this growth.

After considerable study, the Memphis Light, Gas and Water Division adopted a program for the building of generating units sufficient to meet the present and future needs of the City. The Division is managed by the Board of Light, Gas and Water Commissioners, whose president is Major Thos. H. Allen after whom the plant is named.

The adopted program calls for the initial installation of three generating units, each having a nameplate rating of 250,000 kw. The first unit is scheduled for commercial operation late in 1958 and will be followed by the second and third units at two-month intervals. The initial installation will have a gross capability of 862,500 kw, becoming the 10th largest steam plant in the United States. The program adopted provides for adding additional units of like size as needed.

Each of the three initial turbines are 3600-rpm, three-cylinder, tandem-compound, triple-exhaust types. The machines are operated at steam conditions of 2400 psig and 1050° F at the turbine throttle with reheat to 1050° F. The over-all dimensions of each turbine generator are approximately 107 feet in length, 27 feet in width, and 13 feet above the operating floor.

For each turbine generator there is one steam generator. The initial steam generators are cyclone type with a capacity of 2,100,000 pounds of superheated steam per hour at 1053° F and operating pressure of 2475 psig. Each unit will be served by a reinforced concrete stack, 400 feet high. There are no induced draft fans in the system. Electrostatic precipitators will remove fly ash from the exhaust gases.

The steam generators are designed for firing by either coal or natural gas. At peak capacity the three initial boilers will consume approximately 300 tons of coal per hour. Coal will be delivered by barge and the storage area provided is for 90 days' operation, or approximately 450,000 tons.

Site Selection

The work of gathering data for the determination of a suitable site on the Mississippi River was begun in September, 1955. In all, eight individual sites were studied. Four showing the most promise were subjected to rigorous analysis.

Among the many factors which were considered were the following:

1. A continuously open channel of sufficient depth for access of coal barges and the adaptability of the site to installation of docking and unloading facilities.

2. Availability of large quantities of cooling water not affected by recirculation.
3. Subsurface conditions suitable for heavy structures.
4. Access to plant site by road, railroad and river.
5. Topographic conditions as they affect the cost of final grading.
6. Work in progress of work contemplated by others for the stabilization of river banks and maintenance of the shipping channel. The Corps of Engineers has a long-term program under way for such work both upstream and downstream of Memphis.
7. Ash disposal area.
8. Proximity to built-up areas from where labor may be recruited for construction work and later for plant operation and maintenance.
9. The development of a transmission system or connection to an existing system.
10. Ease of acquiring land, land rights and right-of-way.
11. Tax considerations.
12. Public relations.

Of paramount importance, from a civil engineering standpoint, was the accommodation of waterfront features absolutely necessary for the practical and economical operation of a plant of this magnitude. Large quantities of water are required for cooling purposes and a navigable channel is necessary to provide easy access to docking facilities for coal barges.

The Mississippi River provides water in abundance and a navigable channel. However, major problems had to be considered and overcome. The first problem was the variation of the river level, which in the vicinity of Memphis averages 46 feet over the year. The difference on record between extreme high and extreme low is 59 feet.

A second problem is the tendency of the river to create new channels, as evidenced by its historic behavior. In an effort to determine the stability of the channel in any particular location, careful review was given to records assembled by the Corps of Engineers over the last 150 years. During flood seasons, severe scouring of the river bottom or active silting may cause a change in the flow of the river. It was apparent that the site had to be chosen with a view to the satisfactory control of the river problems in order to insure safety and permit continuous, reliable plant operation.

The site ultimately chosen, Ensley, or Ensley's Bottom (Fig. 1), is located in a large area of bottom lands on the south side of McKellar Lake across from Presidents Island. McKellar Lake was formerly the Tennessee Chute, an abandoned Mississippi River channel. The lake was formed by building a causeway between the mainland and Presidents Island, thus closing the north inlet to the Chute. The site is about five air miles southwest of the business center of Memphis. Including accretions, one thousand and one acres were purchased for this installation. The generating station including plant building, switchyard, pump house and coal storage area occupy about 160 acres.

The site has many favorable features. Sheltered by Presidents Island on the upstream side, it is safe against damaging attack by the river. Because it is located within a moderately populated area, a satisfactory labor supply is assured. The area offers easy access to transmission lines and exceptionally long spans are not required. It is near an existing railroad line and highway. Finally, another favorable feature of this site which is essential to coal burning plants is adequate space for the expansion of plant, docking facilities, storage area and ash disposal.

Fig. 1. Thos. H. Allen Electric Generating Station.
Aerial View Showing Site Location.

There were also other features which proved advantageous in the development of the site. As natural harbor facilities did not exist, a channel and harbor for movement of coal barges had to be dredged out of the lake bottom. On the other hand, final grading of the plant area to bring it up to an elevation above any anticipated flood level required large quantities of readily available fill. The dredged material was suitable for this purpose and provided virtually all the fill necessary.

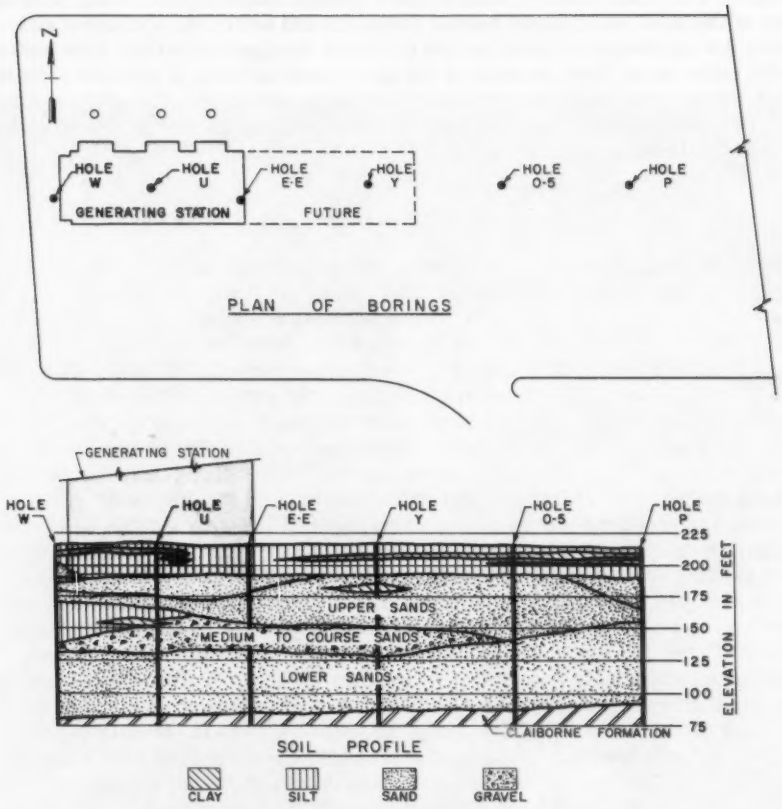
Site Preparation

In order to afford a clear picture of subsurface conditions, a planned program of soil borings was undertaken. The first borings revealed the presence of a large clay lense in the top stratum of the building site being considered. Since this clay would have caused excessive settlement in the fill, and in the coal storage pile, the powerhouse was shifted 2500 feet westward to its present position, where more suitable material was encountered. A total of 26 holes were drilled at the new building location to depths of between 100 and 150 feet. This total does not include any others that were drilled in locations of other structures and facilities. Samples were recovered at intervals of five feet for identification and classification.

From original grade at an elevation of about 219 feet to a depth of 130 feet, borings revealed a river alluvium consisting of silty sands, clays and clean sands varying from very fine to coarse. A soil profile taken through the middle of the building area is shown in Fig. 2. The river alluvium is shown underlain by a series of very dense clays and sands called the Claiborne Formation. These are highly consolidated sediments of considerable thickness in which no appreciable settlement is anticipated as a result of the foundation loads transmitted through the overburden. A study of the physical characteristics and the blow counts of the sampling spoon revealed three distinctive strata in the overburden. The topmost stratum, having a total thickness of about 23 feet, consists of lenses of very fine sands, silts and a range of non-sensitive clays. Below the top stratum to a depth of about 66 feet is a stratum referred to as the "Upper Sands" which consists of dense fine sands, except at the west end of the area where the sands become medium to coarse. Between the "Upper Sands" and the Claiborne Formation is a third stratum referred to as the "Lower Sands" in which are recorded the higher blow counts.

A site plan of the plant building and all the associated works is shown in Fig. 3. The preparation of the site was a challenging problem to both engineers and constructors. Except for those portions reserved for ash disposal, the entire plant area had to be built up an average of 20 feet in order to insure against inundation during the highest flood levels.

Virtually all the required fill material was obtained from dredgings in the harbor development work. The waterfront at the site had an undeveloped shore line and waters too shallow for navigation during all but flood periods. Its conversion to a useful and safe harbor for coal barges depended on an extensive and successful dredging operation. The dredged material composed chiefly of fine to medium sand with varying amounts of silt proved suitable as land fill. It was determined that a one-on-six slope in submerged areas and a one-on-four slope for areas above high water would be more than sufficient against embankment failure. By careful planning of operations, a maximum quantity of fill, amounting to nearly three million cubic yards, was placed by



THOS. H. ALLEN ELECTRIC GENERATING STATION
SOIL PROFILE AT PLANT SITE
FIGURE 2

hydraulic methods. An additional one million cubic yards was placed by mechanical means in areas that could not be conveniently or safely filled hydraulically. Filled areas included not only the building area, but also the road and railroad embankments to a distance nearly two miles from the plant.

Fig. 4 shows a view of the inlet channel and harbor. The harbor spans the full length of the mooring line, about 2500 feet, and the inlet channel extends northward to the south limit of the channel maintained by the U. S. Army Engineers for river traffic into McKellar Lake. The total area developed, about one-fifth square mile, had an average bottom elevation of 195 feet which was dredged to elevation 163 feet.

Excavation for the plant building produced 200,000 cubic yards of suitable fill material and was used to form a dike around the excavation before placing hydraulic fill. In order to enable the work to progress without interruption from water, a complete well-point system was installed capable of dewatering the excavation against a head of 50 feet in the event that the river level rose

THOS H ALLEN ELECTRIC GENERATING STATION
SITE PLAN
FIGURE 3
MISSISSIPPI RIVER
MC KELLAR LAKE
MOORING CHANNEL
INLET
MOORING CHANNEL
COAL UNLOADING TOWER
CONVEYOR BRIDGE
MOORING LINE
TRANSFER TOWER
COAL STORAGE AREA
CRUSHER TOWER
FUTURE
FUTURE
INTAKE PUMP HOUSE
3 UNIT GENERATING STATION
SWITCH YARD
OUTGOING TRANSMISSION LINES
ASH DISPOSAL AREA
MATCH LINE
PROPERTY LINE
PLANT ENTRANCE ROAD
PROPERTY LINE
TUNNEL
DISCHARGE
EXISTING 60" SEWER
CANAL
DISCHARGE
FUTURE
EXISTING 60" SEWER
EXISTING MITCHELL ROAD
PLANT ENTRANCE ROAD
EXISTING R.R.
NEW BRIDGE
NEW TRACK
NEW BRIDGE
HORN LAKE CUT OFF
PROPERTY LINE
FUTURE SEWER
PROPERTY LINE
MATCH LINE
SCALE IN FEET
0 400 800
N

Fig. 4. View of Inlet Channel and Harbor.

to flood stage. Sand fill was stockpiled around all excavations to be used later for backfilling.

Preparation of the site also included maintaining the only existing vehicular access, an old farm road that had been under 15 to 17 feet of water during the 1937 flood. A new road was laid down to join Mitchell Road, a main artery, at a point two miles east of the site. This road is 24 feet wide and has a bituminous concrete pavement. It is at an elevation sufficiently above flood stage to ensure year-round access. Two concrete bridges were installed as part of this road, the first 132 feet long over the Horn Lake cutoff, and the second 139 feet long over the track of the Yazoo & Mississippi Valley Railroad. The filled area for the roadbed was made wide enough to accommodate a new track and a future track joining the Yazoo & Mississippi Valley Railroad at a point about one and three-quarter miles east of the site.

An existing 60-inch-diameter sewer outfall runs from east to west through the property. Where the railroad track and road pass over this line, a concrete relieving platform was installed as a precaution against damage to the sewer.

Two twelve-inch city water mains, one on each side of the road embankment, were extended into the property and joined to form a loop. Temporary electric service for construction work was provided by the installation of a 12,500-volt service of 2450-kva capacity to the site.

Construction buildings containing office, sanitary and storage space could not be erected until the land fill was in place. Therefore, it was necessary to rent trailers in sufficient numbers to serve temporarily. The construction buildings now in place will ultimately become service buildings as part of the final plant installation. The buildings have aluminum siding on steel framing and rest on spread footings.

Temporary auto parking and material storage areas were developed at the east end of the site in the area ultimately intended for coal storage. To provide access to these areas as well as to all sections of the site many temporary roads were developed. A chain link fence was placed around the plant area.

General Plant Arrangement

A composite plan of the plant building taken above the operating floor is shown in Fig. 5. A section through the building is shown in Fig. 6. The plant has a full basement and is fully housed. Operating requirements, as well as natural site conditions, to a large measure, influenced selection of the adopted arrangement.

The great variations in river water level experienced in the Memphis area point to the submerged basement for maximum economies in pumping cost. The ideal arrangement, from this point of view, is to set the top of condenser at an elevation that will permit full recovery of the static head by siphon action even during the lowest river water levels. However, savings in pumping cost must be evaluated in terms of increases in building cost. With natural grade at about elevation 219 feet, maximum advantage of the hydraulic conditions of the site was effected by setting the top of foundation mat at 199 feet. Accordingly, the operating floor is set at elevation 240 feet, or 8.6 feet above the record flood crest of 1937, and the basement finished floor is set at elevation 203 feet.

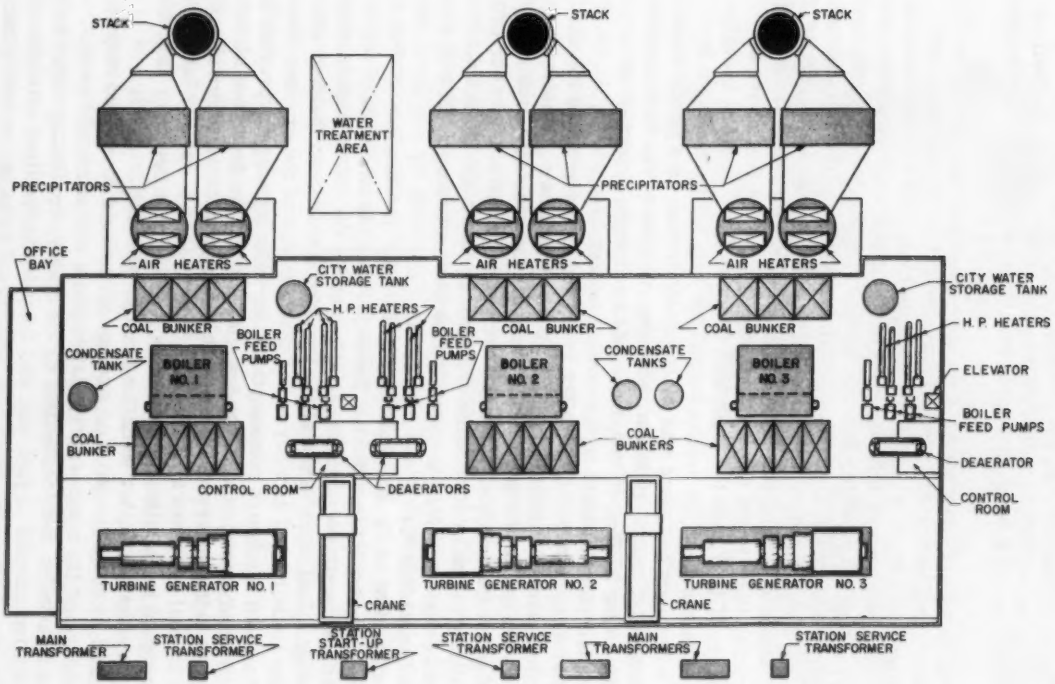
The use of a semioutdoor plant was given consideration as a practical design in the Memphis climate. However, with the adoption of the basement design already providing 37 feet of basement wall enclosure, it proved advantageous to complete the structure as an enclosed plant.

From an operating standpoint, the plant is arranged and oriented for maximum efficiency. Considering the large quantities of coal being received, stored, crushed and delivered to the plant bunkers, the efficient handling of this item is of great importance. Coal is received by barge with access provided to the mooring dock by way of the dredged channel and boat basin. Future railroad delivery of coal is possible, but the necessary facilities are not being installed at this time.

Fig. 7 shows a view of the coal handling facilities. The location of the transfer tower with respect to adjoining facilities was planned to hold conveyer runs to a minimum while still leaving room for the future expansion of the plant building. Coal is received at the transfer tower from the dock unloader and may be delivered directly to the plant by way of the crusher tower or may be stockpiled for future consumption. Reclaiming hoppers provide the means for delivering stored coal to the transfer tower. An area east of the transfer tower is being reserved for a possible future track unloader if railroad delivery of coal becomes an actuality. The crusher tower is centrally located with respect to the ultimate station so that the belt conveyer may move economically in either direction when the plant is extended.

The preferred location for the coal storage area would have been between the dock unloader and the plant, except that to place it there would have necessitated a relocation and lengthening of the costly circulating water intake lines. The location adopted is just east and adjacent to the final extension of the ultimate plant. As present plans call for all coal to be delivered by barge, rail handling facilities have been kept to a minimum. However, space has been reserved in the area southwest of the coal yard for installing several thousand feet of storage track if this becomes necessary.

Ash disposal areas are abundant, as considerable lowland within and without the property will need filling in for future development. The first area to be filled will be that along the waterfront east of the plant.

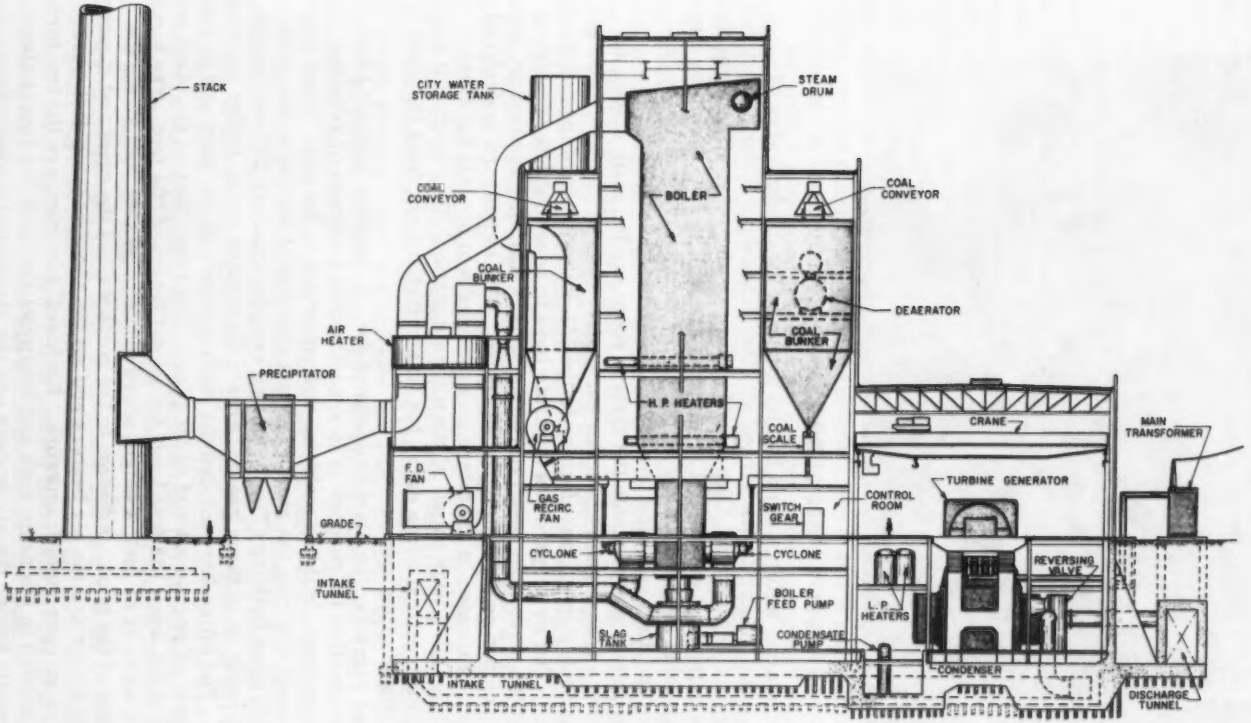


THOS. H. ALLEN ELECTRIC GENERATING STATION

COMPOSITE PLAN

FIGURE 5

FIGURE 5



THOS. H. ALLEN ELECTRIC GENERATING STATION
CROSS SECTION

FIGURE 6

Fig. 7. View of Coal Handling Facilities.

The circulating water system has its intake in the dredged area of McKellar Lake. Water is discharged into the Mississippi River downstream of the entrance to McKellar Lake in order to avoid the possibility of recirculation. Pumping cost and capital investment are kept to a minimum by keeping the runs as short as possible and by the use of an open channel in the last 4400 feet of the discharge line in lieu of a closed conduit.

Located immediately outside the turbine room are the main stepup and station service transformers, one for each generator. A 161-kv circuit extends from each step-up transformer to the 161-kv switchyard located south of the plant. The switchyard has separate bays to accommodate one circuit for each of the three main generators, the station startup transformer, seven outgoing lines and one transfer bus tie circuit. The switchyard is designed so that it may be extended in the future on a unit basis to provide for transmission of the full power output of the ultimate station. Double circuit steel towers with two ground wires carry the transmission lines from the plant to the City's distribution system.

The road servicing the plant is designed for the heaviest highway traffic. During construction its purpose is to accommodate hundreds of vehicles carrying personnel, building material and equipment to the site. When completed the road will ring the plant to ensure convenient access to all areas.

The main plant building (Fig. 5) measures approximately 207 feet by 534 feet and is made up of three general areas; the turbine room, boiler room and office bay. The office bay includes the main entrance to the plant and is located at the west or permanent end of the plant. The remainder of the plant contains three units, each 168 feet long, for a total length of 504 feet. The turbine room has a width of 85 feet, a height above grade of 50 feet, and forms the south portion of the plant. Adjacent to the north wall of the plant and directly behind each boiler are the air heaters and all the accompanying duct work supported on an open steel-framework. Enclosed, however, are the two forced draft fans located in the same area and occupying the first two levels above grade. Directly behind the air heaters are the electrostatic precipitators and the breeching that lead to the individual stacks. Adjacent to and resting on the counterforts of the north wall is the intake tunnel of the circulating water system. This tunnel, acting as a manifold, feeds water into tunnels which pass

under the building mat to serve the individual condensers. On the exterior side of the south wall and resting on the foundation mat is the circulating water discharge tunnel.

The turbine room houses the three turbine generator units and auxiliary equipment. The units are placed with their axes parallel to the long dimension of the plant. With this arrangement the bridge crane and roof trusses have the shortest possible span and optimum use is made of the building space. The plant is equipped with two separate bridge cranes, each of 125-ton capacity and together capable of lifting a stator which is the heaviest piece of the turbine generator. Each crane is also equipped with an auxiliary hook of 15-ton capacity. The availability of the two cranes for the three units of the initial station and the future units is not only desirable from an operation standpoint, but will have effected substantial savings in the erection of all the units. The operating floor, designed to support the heavy loads of turbine parts during assembly and disassembly, has a large open hatch adjacent to each unit and removable floor plates at hatches over condensate pumps, oil coolers, and the reversing valves at the condensers' inlets and outlets. A spare exciter is housed on the operating floor. In each unit the basement contains the turbine foundation, condenser, cooling water piping and valves, turbine oil tank and coolers, condensate pumps and various other related equipment. Alongside each turbine generator unit is a condensate pit 23 feet by 56 feet by 16 feet deep. On the mezzanine platform are heaters No. 2 and 3.

The boiler room contains the steam generating units complete with appurtenant auxiliaries. In general, the layout provides for a symmetrical arrangement around each pair of boilers. For instance, there are two central control rooms, one for Units No. 1 and No. 2 and one for No. 3 and future No. 4. Likewise, for each two units, the arrangement provides for a building elevator, stair, city water storage tank, and several other such items intended to be common to both.

Each boiler is served by two separate coal bunkers with a total storage capacity of 4500 tons. The coal bunkers are located in bays adjacent to the front and rear of the boilers and deliver coal to the seven cyclone burners in each unit by means of separate hoppers, scales and coal feeders. Each bunker bay has an enclosed conveyer gallery above the bunkers. Conveyer belts run the full length of the present unit and new belts will be added over the future bunkers as new units are added. For the initial three units, there is one tripper in each conveyer gallery.

The boiler room basement contains many items of heavy equipment. Beneath each boiler are two slag tanks and related hydraulic ash handling equipment. The tanks are set adjacent to an ash pit built into the foundation mat. Ash sluice piping leaves the plant through space provided in the floor by a trench, three feet wide. There are, also, three boiler feed pumps for each boiler, condensate coolers, service water pumps, blowdown tanks, various chemical pumps and tanks, air compressors, air receivers, motor control panels and electrical substations.

Located on the operating floor at elevation 240 feet are the 4160-volt switchgear, various control panels, boiler forced draft fans, building ventilating fans, control rooms with vertical control board and mechanical and electrical benchboards. Each boiler has a condensate storage tank of 50,000 gallons' capacity supported at elevation 251 feet. Other boiler room equipment above this level include the gas recirculating fans, heaters No. 5 to No. 8 inclusive, and deaerator heater No. 4 with storage tank. On the conveyer gallery roof at

elevation 358 feet are supported two city water storage tanks, each of 70,000 gallons' capacity. At elevation 294 feet are some additional building ventilating fans.

The office bay provides five levels of floor space, three above grade and two below. The main entrance is at grade and includes a lobby and reception area. On floors above grade are located the general offices, assembly room, conference rooms, plant laboratory, first aid quarters and dining area where workers may have their home-prepared meals. The entire space is air-conditioned. A passenger elevator and three sets of stairs provide access to the several floors. Most of the area at the lower basement level is given to the mechanical and electrical machine shops. Each is equipped with a three-ton-capacity electric crane and hoist. The mezzanine level in the basement contains the locker rooms, lavatories and shower rooms for the plant personnel.

The main plant building is ventilated and kept dust-tight by a pressurized system. A total of twelve ventilating fans are located for maximum effect throughout the building. Six fans have a capacity of 220,000 cfm against a pressure differential of 2-1/2 inches of water. The remaining six fans are rated at 145,000 cfm working against a pressure of 2 inches of water. Spent air is exhausted through gravity-type roof ventilators equipped with automatic pressure relief devices.

Among the many other facilities included with the building services are the fire alarm and fire protection system employing water and chemical fire-fighting equipment. Also, there is an intercom system reaching all sections of the plant. There are compressed air systems for soot blowers, instruments, controls and general station service. Hoists are included wherever they are needed through the plant to assist in maintenance operations. There are the fully equipped maintenance shops and laboratories set up for proper supervision and control of boiler make-up water, fuel checks and general plant service. A vacuum cleaning system is provided for general cleanup and dust control.

The external appearance of the main plant building (Fig. 8) is developed along functional lines. The horizontal line is accentuated and its effect is to offset the high chimneys in the background. The exterior walls of the plant are insulated vitreous enameled aluminum siding in a soft grey-green tone. Windows and louvered air intakes are aluminum and are laid out along an unbroken horizontal line. The office bay adds to the effect of the horizontal line with its continuous window wall construction. Windows in the office bay are glazed with heat absorbing glass to help the air conditioning within.

Inside the plant, to a height of fifteen feet above the operating floor, the building walls are finished in ceramic glazed tile. Above the tile the walls are painted steel panels. Below the operating floor walls are all exposed concrete. Concrete floors, in general, have a monolithic finish. The operating floor has quarry tile throughout. Roofs are of flat precast concrete planks with exposed undersides finish painted. Steel framing is exposed except in the office area. Ventilating fan enclosures are of cement-asbestos sheets with metal battens. Control rooms have ceramic glazed units, luminous ceilings, vinyl-tiled floors and air conditioning. The office area is completely finished inside with walls of green ceramic glazed facing tile, floors of vinyl tile and acoustical ceilings with recessed lighting. The entrance lobby has a terrazzo floor and aluminum door and doorframe.

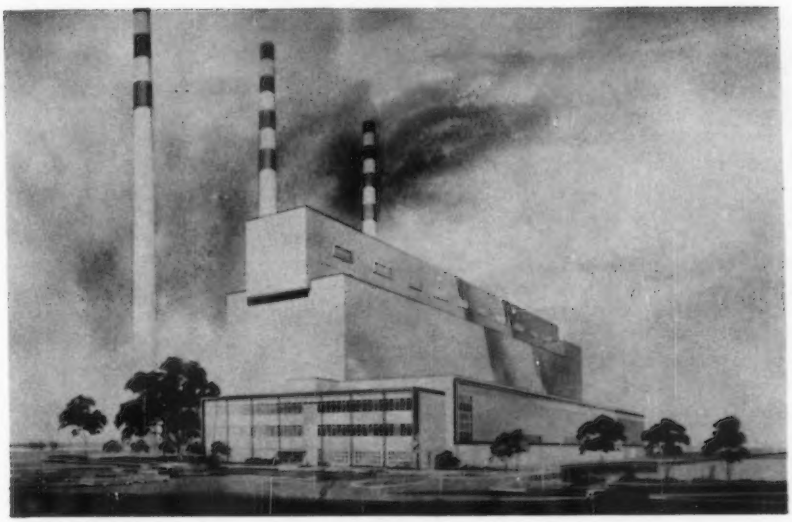


Fig. 8. Perspective View.

Civil Engineering Features

The main plant building rests on a boxlike foundation structure of reinforced concrete (Fig. 9). The bottom of the box is a reinforced slab covering an area of approximately 155,000 square feet and having an average thickness of about six feet. A watertight concrete retaining wall of counterfort construction, 41 feet high, forms the sides of the box and provides the piers on which the exterior building columns are set. The mat and walls are designed to resist the hydrostatic pressures induced by a rise in the river water level to elevation 238 feet, or just two feet below the top of the wall. The plant foundation may be likened to a huge concrete boat which may tend to lift, if not weighted down, during high water periods.

Some thought was given to resting the foundation directly on the underlying soil for its supports. However, it was decided to employ piles for the support of the foundation mat after considering the results of soil borings and tests on soil samples. With the top of the foundation mat set at elevation 199 feet, the immediate supporting soils under the foundation mat, if piles had not been used, would have been those in the lower portion of the top stratum consisting of fine sands, silts and clays (Fig. 2). Analysis indicated that settlements under foundations placed at this level would have exceeded those tolerable in a power plant. A large measure of this settlement would be due to the consolidation of lenses of soft silts and loose sands that occur in the underlying stratum between elevation 140 and 170 feet. In order to obtain a firm support for the structure, it was necessary to use piles driven through this unsatisfactory material and into the firm material generally encountered at about elevation 140 feet.

Piling under the plant foundation, however, was required to perform still another function besides furnishing support for building loads due to the weight

Fig. 9. Excavation and Foundation Work for Main Plant Building.

of the completed structure. The completed building as a whole is not subject to uplift even under the worst possible flood conditions. Nevertheless, during construction and before sufficient building loads could be placed upon it, the boxlike foundation structure might have been subjected to severe uplift forces had a major flood and a breakdown of the well-point system occurred at the same time. Provisions to flood the basement during high water, while offering positive protection, would have meant lost construction time and possible damage to installed work. It was decided, therefore, to depend on the piling to insure against uplift. As piling can resist uplift only through skin friction, the conditions under which such skin friction could be safely relied upon were analyzed. It was decided not to place any reliance on skin friction developed in soils within the range of ground water fluctuations, that is, on that portion of the pile above elevation 170 feet.

A pile testing program was undertaken to determine the safe bearing value of piles driven into the upper part of the "Lower Sands" stratum. It was also necessary to develop the safe pull-out resistance that could be developed by these piles. About 20 piles were tested. Represented among the piles tested were the 14-inch-square precast concrete pile, and cast-in-place, thin-shelled piles which prospective bidders offered for consideration. Test piles were driven to about elevation 135 feet and subjected to both positive and negative load tests. Also, short piles driven to elevation 170 feet were subjected to pull-up tests for the purpose of determining the amount of resistance developed in the soils subject to ground water fluctuations. The safe bearing value and resistance to uplift of the selected long piles was established by discounting the resistance developed in the short piles.

The results of the pile testing program were completely satisfactory, indicating that piles of 50-ton bearing capacity could be developed with

consistency when driven a short distance beyond elevation 140 feet. Pull-out tests indicated that a design based on 25-ton resistance would have a factor of safety greater than two.

The pile used under the foundation of the main plant is the 12-inch-diameter cast-in-place pile with corrugated metal shell. A general view of the pile-driving work is shown in Fig. 10. The design mix of the concrete fill is 4000 lb per sq in. at 28 days. The total number of piles placed under the main plant building was 6301 having an average length of 60 feet. As some difficulty was encountered in driving piles through the upper sands, a limited amount of jetting was permitted.

The foundation piles receive their loads from the reinforced concrete foundation mat. It is designed as a continuous slab transmitting to the piling all loads from building columns, basement walls, turbine generator and all other equipment foundations, circulating water conduits, and the dead and live loads of the basement floor itself. In addition, the mat is designed to resist uplift due to high water conditions in the river during and after construction. It is secured to each pile by reinforcing bars embedded into the concrete of the cast-in-place piles. The mat has a minimum depth of 4 feet 8 inches and is thickened up to a maximum of 8 feet 8 inches beneath the boiler columns, turbine generator foundations and at the circulating water tunnels. The design is based upon a live load on the basement floor of 500 lb per sq ft, the dead load of the mat, three and one-half feet of sand fill and a finished basement floor slab of six inches. The maximum column load is 4500 kips.

Design conditions for the basement provide for a flood stage to elevation 238 feet which is approximately five feet above the previous maximum flood stage established in 1937. Basement walls have been designed to resist an equivalent fluid pressure of 85 lb per sq ft for the saturated earth. The walls are supported by counterforts, spaced twelve feet on centers. Counterforts are 34 feet high and have a base 15 feet long.

For the mat and walls, concrete with an ultimate strength of 4000 lb per sq in. at 28 days was specified. The walls were poured in alternate sections not exceeding 36 feet in order to reduce the possibility of shrinkage cracks. All construction joints are made watertight by the use of metal water stops. The walls were poured in one lift, 41 feet high, and the placement of concrete proceeded at a controlled uniform rate to avoid overstressing the forms. The total volume of concrete placed for the construction of the foundation mat and walls was 53,250 cubic yards.

The foundations supporting the turbine generator units rest on the foundation mat to which they are shear-keyed and doweled. Their design is governed by limited deflections under the normal operating and maximum torque conditions stipulated by the equipment manufacturer. The deflection under any one support is less than the maximum allowable deflection of .02 inches. A total of 1906 cubic yards of concrete was used in the foundation under each unit.

The building framework is of exposed steel. The heaviest loads on the framework are due to the boiler, coal bunkers and city water tanks. Other loads which the structure is designed to sustain are the dead loads of the structure, roof and floor live loads, equipment weights, impact loads, wind and seismic loads.

Full live loads were included in the design of affected members of the structure except that some reductions were used in the design of columns and

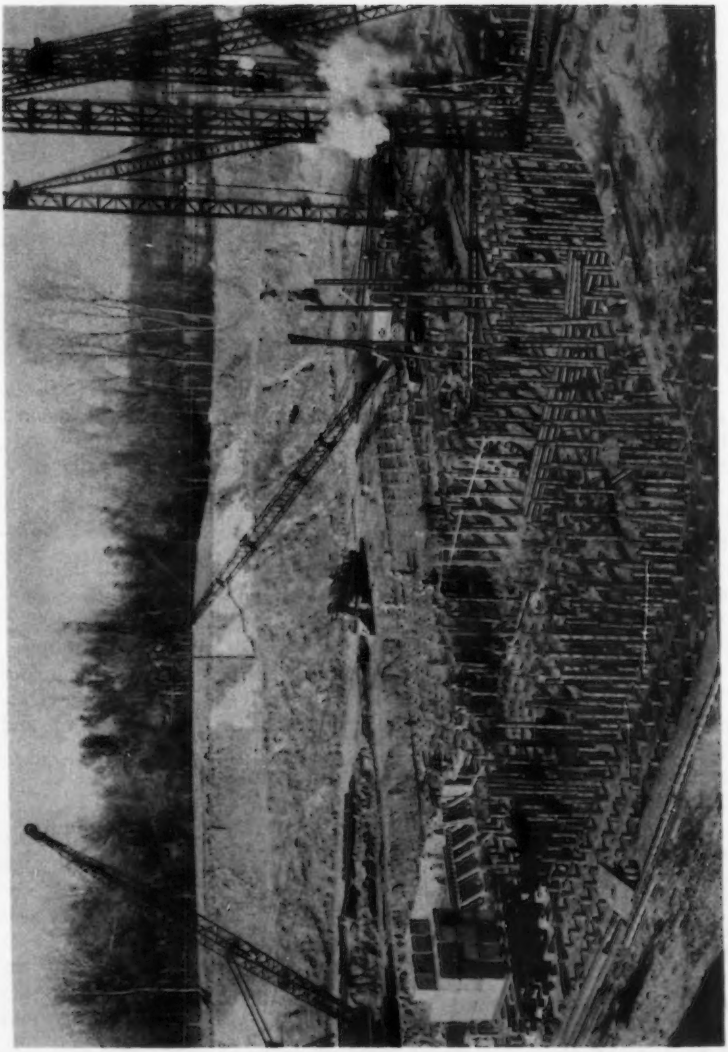


Fig. 10. Placing Piles for Main Plant Building.

foundation. The live loads and the percentage reductions assumed are as follows:

Source	Loading (lb per sq ft)	Percent Reduction
Roof	40	None
Operating Floor	350	25
Intermediate Concrete Floors	100 to 200	25
Grating Platforms	100	25
Equipment and Piping	Given or assumed	None

Because of the accelerated design and construction schedule and because of the lead time required by the mills taking orders for structural steel, it became necessary during the design effort to approximate the weights and method of supporting equipment not yet purchased. Loads due to piping were so large and concentrated in some areas that the usual assumption of an equivalent uniform load could not be reasonably applied. Here again, loads and points of applications had to be assumed in order to complete the design in the time required. To do this with any success, it became necessary to make a preliminary piping layout including hangers and anchors. As final equipment and piping loads became available, the affected areas were rechecked. Some modifications after fabrication were necessary, but the schedule of steel erection was maintained to the advantage of the whole program. A view of the steelwork for the main plant building is shown in Fig. 11.

The plant is located in a Zone I earthquake intensity area as defined by the Uniform Building Code. The design, therefore, includes analyses of lateral bracings for the applicable forces due to seismic shock loads applied at the

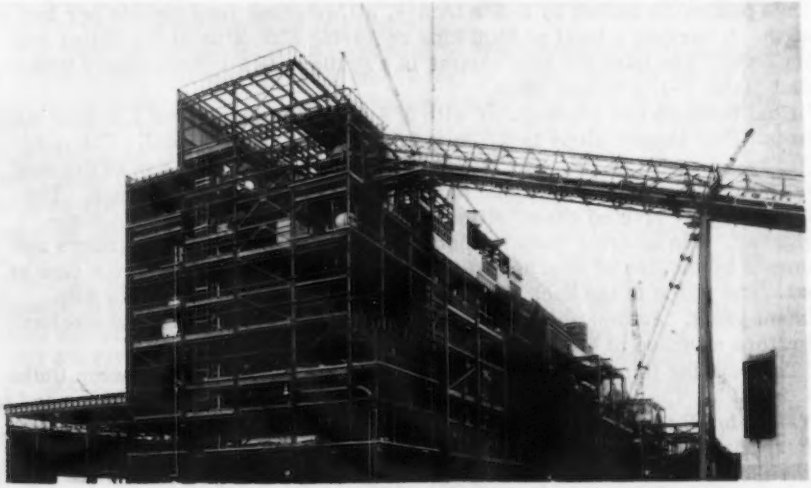


Fig. 11. Steelwork for Main Plant Building.

centers of gravity of the various portions of the structures. A wind load of 25 lb per sq ft was assumed to act on all surfaces exposed to wind except for the coal handling structures for which a 30 lb per sq ft loading was assumed.

The design of steel supporting fans allowed for impact by assuming double the weight of fan and motor. Floor slabs at the fans are thickened to weigh two and one-half times the weight of fan and motor. In addition, stresses in steel beams were limited to 12,000 lb per sq in. and beam depths held to about one tenth the span.

The crane girder is a wide flange section with a channel connected to the top flange. The member is considered to act as a composite beam section designed to withstand impact and thrust load factors for overhead electric-operated cranes as designated by the American Institute of Steel Construction Specifications.

The boiler house framing is built around the boilers. The boiler columns form a tall tower from which the boiler is hung from support steel 200 feet above the foundation mat. An interesting feature is the system of cross-bracing between the boiler columns, which combined with the walls of coal bunkers in bays adjacent to the boiler provide stability and resistance to wind and earthquake forces. By this method heavily framed moment connections are held to a minimum.

In the turbine room, there was a substantial savings in structural steel in the crane columns and girders due to the two-crane installation as compared to a single-crane installation of equal capacity. The turbine roof is supported on steel trusses 85 feet in length and spaced 24 feet on centers. They were shipped and erected in single units.

All shopwork is riveted or welded. Field connections are of high tensile strength bolts except bunker supporting girders which are riveted. Inspection specifications of bolted field connections required that at least ten per cent of bolts in any one connection, but not less than one in each gage line, be tested by a calibrated torque wrench. The largest column section is one of those supporting the boiler. It is built up from a 320-pound 14 WF core section with flange plates, 30 inches by 3-5/8 inches, all weighing 1060 pounds per foot of length. It carries a total of 4500 kips receiving 3500 kips at the boiler support level. The base for this column is a grillage of I-beams topped with a billet plate, 9-1/2 inches thick.

Coal bunkers are rectangular with 3/8-inch side plates and 1/2-inch hopper plates. Side hopper slope is 60° to insure a steady flow of coal. The minimum valley angle is 55°. The bunker design is based on the use of crushed coal weighing 50 lb per cu ft and having a maximum size of 3/4 inch. The bunkers are of riveted construction with all joints seal welded for dust-tightness. The interior construction provides for protection of girders and columns by the use of attached wear plates shaped to streamline the flow of coal. The space in the back of the wear plates is completely filled with cement grout. Automatic indicators record the height of coal, and mechanical vibrators attached to the hopper assure continuous flow of coal.

The building is provided with a transverse expansion joint between Units No. 2 and 3. The joint is formed by installing a double row of columns and is carried through outside walls, roof and all floors. Its design is based on a differential temperature of 80° which will cause a change in length of 1-7/8 inches for each two units. Actually the joint will allow a maximum movement of 3 inches. No appreciable change in temperature is expected in the concrete foundation mat as it is protected by the building and is in constant contact with

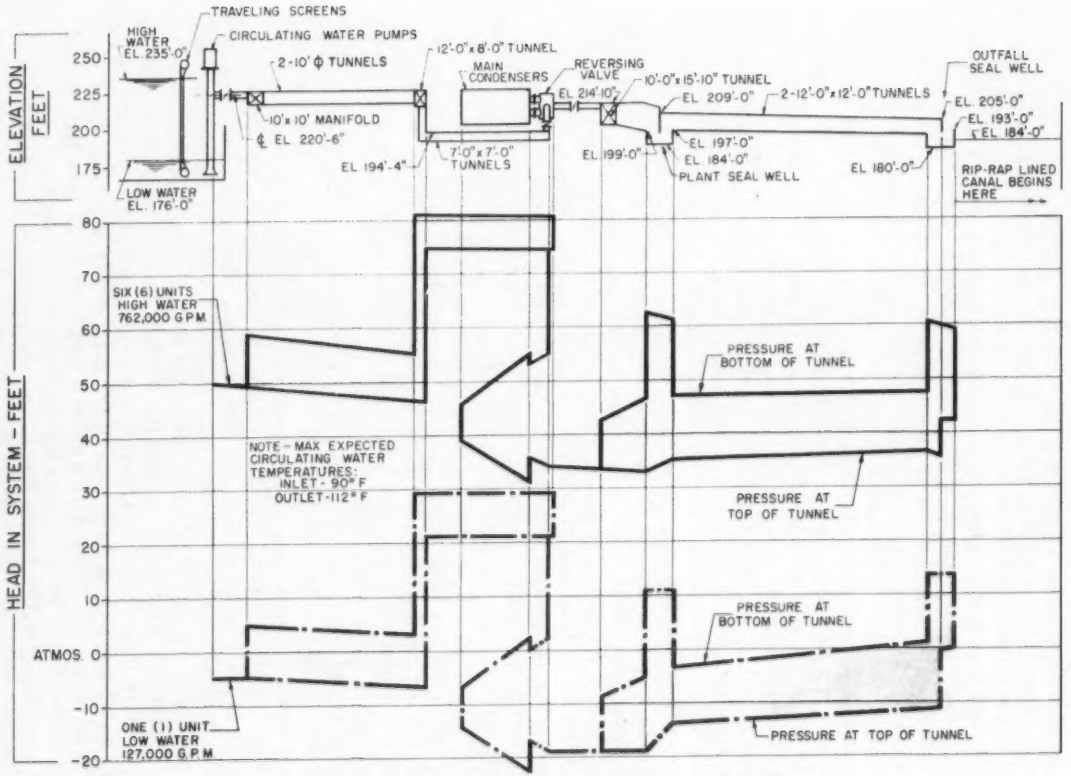
the ground. As some of the basement walls are exposed, the expansion joint is extended below grade to a depth of 16 feet where it is terminated. The joints in the walls are formed with a dumbbell waterstop and compressible filler.

The circulating water system consists of a pump house and intake structure located on McKellar Lake, an intake conduit of twin circular tubes about 600 feet long between the pump house and plant, and an outfall discharge line from the plant to the Mississippi River consisting of a rectangular twin conduit about 4000 feet long discharging into a riprap-lined canal about 4400 feet long. The entire length of run from point of intake at McKellar Lake to point of discharge into the Mississippi River is approximately two miles. The layout of the system (Fig. 3) insures against recirculation as the discharge is well downstream of the inlet to McKellar Lake. The system features utilization of a siphon up to a maximum height of 27 feet when the river level falls below elevation 196 feet as occurs 50 per cent of the time. Essential to the siphon system is the outlet seal well located at the end of the outfall discharge conduit just before it empties into the open canal. A second seal well is built into the discharge end of the plant discharge manifold and serves to protect the siphon system should a break occur in the long outfall discharge line.

Cooling water requirements for the initial installation consisting of three generators will be 380,000 gallons per minute. Six 1250-hp pumps capable of delivering the required amount of water against a head of 57 feet are supported on the pump house deck, also referred to as the operating floor. The total head may reach a maximum at lowest river levels of 81 feet of which 44 feet represent the static head and 37 feet the frictional losses. With a siphon of 27 feet operating, the pumps will work against an effective head of 54 feet. However, during high water levels the static head is greatly reduced and the use of only three pumps will provide the necessary supply of water to the three condensers. In Fig. 12 the pressure gradients for the extreme conditions of operation are illustrated. The curves are based upon quantities of water that will be required with the inlet temperature of 90° F. High water conditions are shown for a possible future six-unit operation. Low water conditions are shown for a single-unit operation.

The pump house and intake structure is of reinforced concrete measuring 68 feet in width, 130 feet in length and 79 feet in depth. Fig. 13 shows the pump house under construction. An exposed operating deck is at elevation 238 feet which is about three feet above maximum flood stage. The floor of the structure is the foundation mat, six feet in depth, resting on cast-in-place piles driven deep for bearing and pull-out resistance. Approximately one half the piles are installed with a batter to offset the thrust of the backfill from the land side. Sheet pile wing walls protect the dredged inlet channel from filling in. The structure includes three separate chambers each containing two circulating water pumps and three sets of traveling water screens. Each chamber is provided with three sluice gates 9 feet wide by 15 feet high so that pumps and screens may be dewatered for maintenance and repair.

All six circulating water pumps discharge into a common manifold having a 10-foot-square cross section running the full length of the pump house. Connected to the manifold are the twin circular tubes of the plant intake conduit, each 10 feet in diameter (Fig. 14). The conduit is designed for a positive water pressure of 113 feet above the crown and a negative pressure of 34 feet of short durations as would occur should the water columns separate. The



THOS. H. ALLEN ELECTRIC GENERATING STATION
PRESSURE GRADIENTS IN CIRCULATING WATER SYSTEM

FIGURE 12

Fig. 13. Pump House Under Construction.

intake conduit rests on bearing piles its full length, as otherwise the tubes would settle under the influence of the fill placed over them.

The intake conduit from the pump house is connected to a plant intake manifold, eight feet wide by twelve feet high, and rests on the counterforts of the north wall. From this plant intake manifold, water is delivered to the condensers by means of seven-foot-square concrete tunnels in the plant foundation mat. After circulating through the condensers, used water is discharged through the reversing valves into a plant discharge manifold, sixteen feet deep by ten feet wide, which is the beginning of the main discharge tunnel. The discharge manifold rests on the mat adjacent to the south foundation wall of the plant. In this tunnel water flows along the south basement wall to a concrete seal well at the southwest corner of the building (Fig. 15). Up to this point the entire circulating water system is supported on piles and is free from any possibility of excessive differential settlement that may cause leaks. Beyond this point the 4000-foot-long outfall discharge conduit rests for support on the natural subgrade.

The outfall discharge conduit (Fig. 16) is of poured concrete with a twin tunnel section, each tunnel 12 feet square. The outlet seal well is located at the end of this line. In order to have maximum efficiency in the system, the outfall conduit must also work as part of the siphon system. This means that it shall be free of air leaks and that the outlet seal well shall function as a barrier to atmospheric pressure. With the tunnel resting directly on earth and surcharged initially with 25 feet and ultimately with 37 feet of fill, some differential settlement is expected. The line has been made flexible to take this movement by providing expansion joints at intervals of 50 feet. The joints are formed to allow an angular movement of six inches in 50 feet. The connection of the discharge conduit to the pile-supported seal well at the plant is made flexible and airtight by a continuous diaphragm seal at the sides and top of the tunnel. The bottom edge of the tunnel rests on and is connected to an extended shelf of the seal well by means of a fabricated steel rocker plate designed to permit vertical angular movements only. An unforeseen break in the long flexible section of the discharge tunnel may cause a loss of the outfall seal. However, the seal well at the plant will maintain the vacuum and save nearly all the 27 feet of siphon head in the system.

PRESSURE GRADIENTS IN CIRCULATING WATER SYSTEM
FIGURE 12

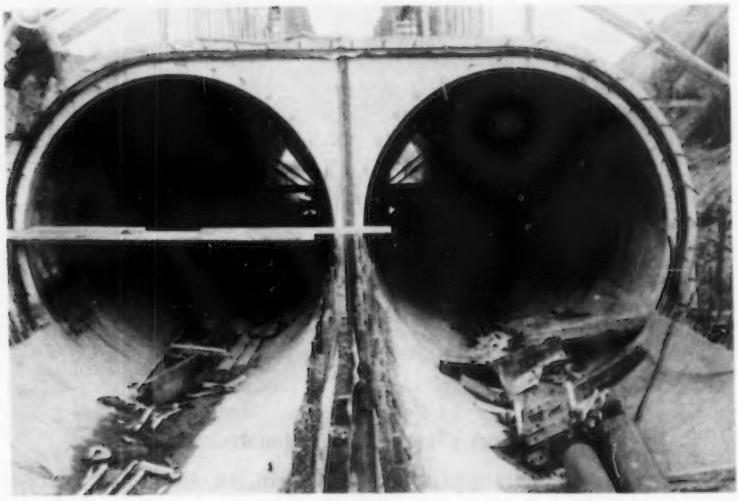


Fig. 14. Intake Conduit from Pump House.

The outfall discharge canal has a flat base, 30 feet wide, and sides sloping 1 on 2 to natural grade. The lining is built of 10-inch riprap placed on a sand bed having a minimum thickness of six inches. The riprap is extended to cover five feet of the bench on each side. Sand drains have been installed along both sides of the canal to prevent the accumulation of water that might damage the walls of the canal. The sand drains are twelve inches in diameter, twelve feet apart and extend to a sand stratum below the canal bottom. The mouth of the discharge canal is stabilized with two six-inch concrete mattresses resting on a six-inch crushed stone base.

Automatic devices have been provided to protect the system from damage due to water hammer. Vacuum breaking valves have been installed along the entire system at all points of potential water column separation. The valves are set to admit air instantly into the system if the vacuum at any point drops below its normal operating pressure. Air that is so introduced will act as a cushion to absorb the impact of the converging water columns. An air removal system is also included to bleed off extraneous air that will have found its way into the system.

The circulating water intake is at the north side of the pump house, which is faced with a steel trash rack measuring 73 feet in height by 130 feet in length. The rack is maintained free of debris by a mechanical rake controlled from the operating deck. A 30-ton gantry crane with a 5-ton auxiliary hoist rides on the deck of the pump house and will be used for handling all equipment. The pump house is designed for stability against uplift in the event that any one of the chambers is dewatered for maintenance work. The east wall is provided with dowels and shear keys for a future addition. An ample number of sluice gates are located throughout the entire system to allow for quick isolation and service to any portion requiring it.

Facilities for handling coal barges (Fig. 4) consist of the dredged harbor and an eighteen-cell dock. The dock has sixteen cells 23 feet in diameter, and two cells 53 feet in diameter, spaced and positioned to form a tangent

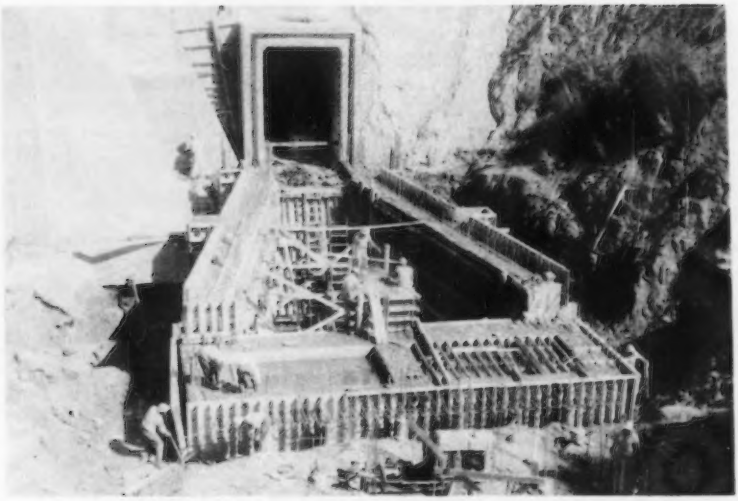


Fig. 15. Seal Well at Southwest Corner of Building.

mooring line 2100 feet long from the first to the last cell. The unloading facilities are at the center of this line. The loaded barges are moored on the downstream end of the line and are moved into position for unloading by hauling lines powered by boat pullers located in the unloading tower. The hauling lines are run through sheaves at each end of the mooring line. As the unloading operation proceeds, the barges are pulled forward and empty barges are moored at the upstream end of the mooring line.

The mooring cells of 23-foot diameter are constructed of interlocked sheet piling driven into the riverbed, filled with sand and capped with a heavy concrete slab. Sheeting is driven 33 feet below the new riverbed which is approximately 30 feet below the original level. The top of cells are above the maximum expected flood level. The total length of each spliced sheet is 105 feet.

The coal unloading tower is a steel frame structure with a height above the top of its supports of 86 feet. It has two trussed booms at the top of the tower extending 90 feet beyond the base of the tower. A 12-ton bucket is operated from each boom having a reach of 89 feet and a vertical lift of 120 feet. The arrangement permits the simultaneous operation of both booms on a single large-sized barge or on two smaller barges when lashed side by side. The unloading tower is located over the two mooring cells of 53-foot diameter which are spaced 56 feet on centers and joined by an interconnecting cell. Supports for the tower consist of steel H-piles driven deep into firm sand and gravel. A concrete slab, five feet thick, caps the sand-filled cells. The deck is 5-1/2 feet above maximum flood stage. The tower is reached from the shore by a bridge 366 feet long and 8 feet wide. The bridge is supported on steel H-pile bents 61 feet on centers. A concrete deck is supported on two steel girders.

The coal conveying system consists of belt conveyers housed in galleries connecting the several towers and handling facilities. The system is capable

Fig. 16. Outfall Discharge Conduit and Rip-Rap Lined Canal.

of delivering up to 2200 tons of coal per hour to the boiler plant or to the coal storage yard stacker by way of the transfer house.

The coal to be stockpiled is conveyed to a stationary stacker tower and chuted to the pinned end of a 148-foot-long movable inclined stacker. The stacker is supported by a steel bent which rides on a curved rail with a radius of 65 feet from the stacker tower. The stacker can be swiveled through an angle of 114° and can store up to 7500 tons of coal in the initial pile from which bulldozers and scrapers move material for spreading and stockpiling in the coal yard.

Two reclaiming hoppers, 43 feet long, 20 feed wide and 26 feet deep, provide the means for obtaining coal from the stockpile. The roof over the hoppers is covered with steel grating and is capable of supporting a loaded power scraper weighing 55 tons. Conveyers from the reclaiming hoppers are housed in inclined tunnels for part of their length and deliver coal to intermediate transfer towers, one for each hopper. Conveyers connect each intermediate transfer tower to the main transfer tower. The main transfer tower is 32 feet long, 27 feet wide and 90 feet high.

From the transfer tower coal is conveyed to the crusher house which is 60 feet by 30 feet and 110 feet high. The crusher house supports two crushers each capable of handling 600 tons of 2-inch coal per hour. In the event of future plant expansion, space is available for the addition of two more crushers. Conveyers from the crusher house transport the coal to two conveyers in the boiler house, one over each row of coal bunkers.

The coal handling structures are covered with protected metal siding and are designed for a lateral wind pressure of 30 lb per sq ft. Earthquake forces were figured as a horizontal force equal to 10 per cent of the vertical load and acting through the center of gravity of the load. The structures are supported on piles, and, in addition, reclaiming hoppers and tunnels are anchored

to piling to resist possible uplift during severe floods. Conveyer galleries are fixed at one end and made free to expand at the other end. Intermediate bent supports are pin-connected to the conveyer bridge trusses to permit movement due to temperature changes.

Ash removal is effected by means of a hydraulic ash disposal system. Ash sluice piping leaves the plant through two concrete trenches fitted with precast concrete covers. The trenches vary in width from 2 feet 8 inches to 3 feet 3 inches and extend 430 feet to the north edge of the filled plant area. At this point ash sluice piping is exposed and supported on concrete pipe supports for a distance of about 2000 feet to the ash disposal area just east of the coal pile. Pipe supports are at every 16 feet and pipe anchors at every 200 feet and at all bends.

All three stacks serving the plant are identical. The stacks are of reinforced concrete and have openings at the top of 12 feet 9 inches, designed for an exit velocity of 100 feet per second. The outside diameter at the base is 30 feet. Wall thickness of the concrete is six inches at the top, increasing to nineteen inches at the bottom. Each stack was erected in 53 lifts averaging 7-1/2 feet per lift and requiring a total of 1000 cubic yards of concrete. The lining is acid-resisting brick 4 inches thick, except at the breeching opening where it is increased to eight inches. The space between brick and concrete is filled with two inches of fiber glass packing and one-quarter-inch acid- and moistureproof coating applied to the interior face of the concrete. Openings for breeching are two in number, 31 feet high by 10-1/2 feet wide. A brick-lined hopper at the bottom collects residual fly ash from the discharged gases. A hydraulic removal system carries off the accumulated fly ash. The stacks are provided with aircraft obstruction lights and 360° platforms for servicing the lights at the top and at the 270-foot level. In addition, the top portion of the stacks are painted alternate bands of white and orange, each 19 feet in width. Vertical ladders with rest seats every 30 feet run the full height of the stacks. The chimney caps are of stainless steel. Except for the painter's trolley which is bronze, miscellaneous steel is copper bearing.

The stacks were erected on reinforced concrete foundations requiring 1500 cubic yards of concrete. Both battered and vertical piles were employed in order to support the dead loads, wind and earthquake forces. The average length of the 261 piles supporting each stack was 80 feet. The selected pile was a 21-inch-diameter cast-in-place thin-shelled corrugated section, 50 feet long, combined with a spiral welded pipe, 30 feet in length.

The electrical switchyard is built on an area measuring 170 feet by 600 feet immediately south of the plant building. Because of the relatively light loads involved, all supports are placed on spread footings. Due to the nature of the material and the method of filling, no appreciable settlement is expected. The area is topped with crushed stone and the subgrade is sloped to drain. Perforated concrete pipe placed in sand trenches collect and carry off the surface waters.

Many other structures are also supported on spread footings. They include the mat for support of the water treatment equipment, the various service buildings, gatehouse, and storage buildings for carbon dioxide and hydrogen bottles.

CONCLUSION

The planning and installation of this large-scale modern power generating station for the City of Memphis demonstrates the need for seeking and developing within reasonable costs certain basic features essential to the successful construction and operation of the plant. Among these features are the ability to install a firm foundation, provide ample cooling water and develop the means for receiving the large quantities of fuel required.

Modern construction methods and equipment have narrowed the gap between the seemingly impossible problem and its satisfactory solution.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

MULTIPLE PURPOSE POWERPLANT CAPACITY

Richard E. Krueger[1]

SYNOPSIS

This paper outlines the major factors establishing the total plant capacity of hydroelectric plants attached to multiple purpose projects of the Bureau of Reclamation. These plants are required to produce saleable power from the scheduled releases under the head available without detriment to primary purposes of the project. The revenue from hydroelectric power is often a major portion of the income required to achieve the payout of the entire project required by Reclamation policy.

There are also included the basic factors to be considered in determining the number of units into which that plant is to be divided.

INTRODUCTION

The generation of power in the large multiple purpose projects such as constructed in the western United States by the Bureau of Reclamation, is a product of the storage and controlled release of river water to satisfy downstream water demands. The revenue from the sale of that power often constitutes a major portion of the money paid into the United States Treasury as repayment of the original appropriation. Operation of Reclamation multiple purpose projects is in contrast to single purpose hydroelectric projects, whose prime objective is the production of saleable power. These single purpose projects operate in a manner that will maintain a nearly constant head with water releases controlled to satisfy power demands.

In developing power on multipurpose projects, the basic parameter establishing the size of the powerplant is the rate of flow required to meet demands for water. The number of units into which that plant is divided lies

Note: Discussion open until January 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2132 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. PO 4, August, 1959.

1. Mech. Engr., Dept. of the Interior, U. S. Bureau of Reclamation, Div. of Design, Hydr. Machinery Branch, Office of Asst. Commr. and Chf. Engr., Denver, Colo.

between that established by the maximum size the manufacturers can build and transport to the site, and that resulting from the minimum size that can be economically and adequately serviced and repaired.

In return, the hydraulic limitations of the turbine may require minor modifications of reservoir operations, and market demands will often justify re-regulation of flow through downstream storage. The sale of this bulk power is usually best made through existing power networks whose power supply comes from many plants, including some whose prime movers are other than hydraulic turbines.

Each multiple purpose project is a new problem and the range of head required may be anything from constant to the maximum that the turbine can accept. Similarly, the flow requirement may range from constant to daily or even hourly changes. The procedure herein discussed is, however, an overriding principle to be added to the normal procedure of reservoir studies and turbine selection(1) when applied to the multiple purpose project.

River Basin Planning

Objectives

The prime objective of river basin planning is to so control the flow in the river that maximum benefits accrue to the people of that area and to the country as a whole. Planning studies usually include irrigation, power, flood control, municipal and industrial supply, navigation, public health, fish and wild life and recreation. Any of these must have demonstrable benefits exceeding the cost of their inclusion in the project before they may become a part of the plan. Some state laws give domestic use such as municipal supply first priority, and agriculture a preference over power and industrial use. However, in most storage reservoirs it is feasible to develop water power, and in the interest of conservation of natural resources, provision for its development should be considered.

Irrigation

The western United States needs a continuous expansion of its farm lands to support its growing economy. In the United States as a whole, the three million per year population increase coupled with an annual loss of nearly two million acres of farm land to urban development will soon wipe out our present surpluses of certain agricultural commodities. New irrigated land from Reclamation Projects will have little effect on these crop surpluses, however, as only a very small part of the irrigated crop falls in that category. Rising construction and operating costs of the justifiable irrigation projects which remain to be developed, can seldom be fully paid for from farm incomes alone, and hence these projects often need the supplemental revenue from power to achieve payout.

The need of power revenue was vividly illustrated in the Colorado-Big Thompson diversion project. The water from the Colorado River watershed on the western slope of the Continental Divide was urgently needed to supplement the water available to the highly developed South Platte River Valley on the eastern slope. The estimated cost of a single purpose project to provide only irrigation water was beyond the ability of the water users to pay. However, by making the project multiple purpose with plants at Green Mountain

Dam in western Colorado and on the eastern slope, sufficient additional revenue was made available to put the entire project on a sound financial basis. These plants utilized the imported water plus some eastern slope water, and the 2,600 feet of head available between the tunnel and the eastern slope storage reservoir for power generation to create the additional revenue.

Flow Regulation

Cyclical and seasonal readjustment of flow for both irrigation and power features in river basin development is illustrated by the Colorado River system which includes the Hoover, Davis, and Parker Powerplants together with Glen Canyon now under construction and other proposed upper Colorado River plants. Short term re-regulation to improve the power potential without detriment to irrigation is illustrated by the paired plants such as Shasta-Keswick, Seminoe-Kortes, and Mary's Lake-Estes.

Available Power

Reservoir Capability

The production of power at a given reservoir site is dependent upon the rate of discharge and the head under which it takes place. The rate of discharge basically determines the physical size and cost of the plant, since the size of the water passages involved is the largest physical features to be enclosed by the structure. This holds true especially in the relatively narrow range usually covered by comparative cost estimates of a given project. The head on the plant establishes the basic type of runner to be used in the plant, and the capacity of the attached generator limits the plant capability to produce power.

Turbine Capability

The Francis type turbine which is usually encountered in multipurpose projects, when operating at synchronous speed is basically a constant head, constant load unit, and any deviation from that condition will result in a lower efficiency. In addition, a change in head results in both a change in the discharge capability and in the available power as illustrated in Figure 1. The full gate power at constant speed varies directly with the head, and the discharge with the square root of the head above the pumping head of the runner. This pumping head is the back pressure developed by the spinning runner, similar to the shut-off head of a centrifugal pump, and varies between 25 percent and 40 percent of the design head for Francis turbines. Field experience has shown that slower speed Francis turbines whose specific speed is less than:

$$n_s = \frac{632}{\sqrt{H}}$$

may be operated under as high as 125 percent or as low as 65 percent of their best efficiency head without serious cavitation or vibration. Over this maximum feasible range of head the full gate discharge will increase to more than 150 percent of that at minimum head and the power capability will increase approximately 350 percent, except as limited by its attached generator. Higher speed units have been limited to a maximum departure of 10

TYPICAL FRANCIS TURBINE PERFORMANCE
CONSTANT SPEED AND FULL GATE
FIGURE 1

percent from the best efficiency head of the installed unit to avoid excessive damage.

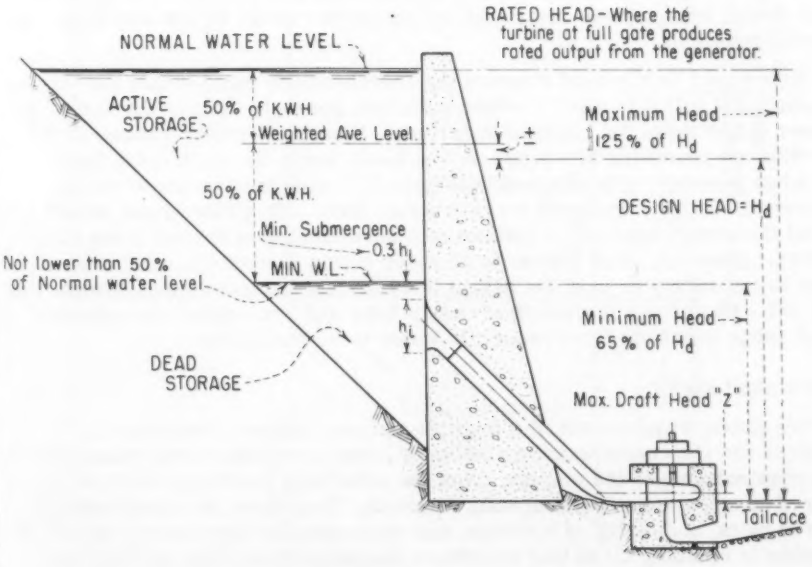
Reservoir Limitations

Reservoir operating studies of multiple purpose projects which include power generation must restrict the variation of water levels during that power generation to the range usable by the turbine. When the active storage contained between these limiting water levels is less than that required to meet water demands, it may be necessary to raise the height of the proposed dam to increase storage or restrict the power production to this limiting range. This modification of reservoir studies also affects the amount of kilowatt-hours generated and, therefore, revenue earned during the average year.

Water Levels and Heads

The several levels and heads significant in power studies are diagrammed in Figure 2 and are defined as follows:

Normal water level is the highest elevation of water level that can be maintained in the reservoir, either with a gated or ungated spillway, without spillway discharge. When the spill is occurring, a surcharge may be imposed



WATER LEVELS AND HEADS
HYDROELECTRIC POWER PLANTS
WITH FRANCIS TYPE TURBINES
FIGURE 2

TYPICAL FRANCIS TURBINE PERFORMANCE
CONSTANT SPEED AND FULL GATE
FIGURE 1

above normal water surface depending on spillway characteristics. However, as spill occurs the tailwater level will usually rise sufficiently to decrease the gross head on the turbine.

Minimum water level for power production is that water elevation producing minimum net head on the units. In multiple unit plants supplied by a single conduit this is the level with one unit operating at full gate. When further drawdown is made thru auxiliary outlets, power production must cease.

Weighted average water level of the reservoir is that level above and below which equal amounts of power are developed during an average year. This level establishes the average head at which the yearly water release will produce a power equivalent to the average yearly power production of the detailed reservoir power studies.

Design head is the net head under which the turbine reaches its peak efficiency at synchronous speed. Here the water velocities and angle of flow relative to the runner are at the values equivalent to design conditions, hence the term design head, or in reference to performance, the best efficiency head.

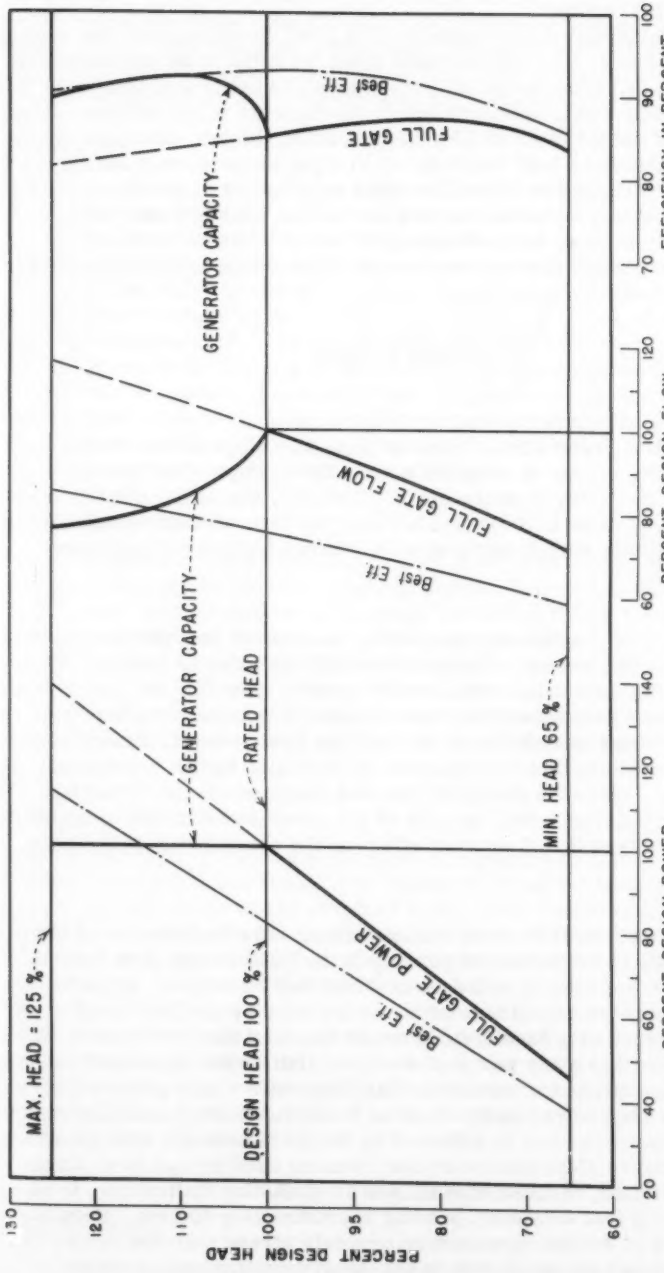
Maximum and minimum heads are the net heads on the turbine establishing the limits of head range, either from reservoir limitations or from turbine capability. With the design head assumed to be 100 percent, the permissible operating range of the turbine will be from 125 percent to 65 percent of that design head, unless restricted to a narrower range by the reservoir water level limitations.

Rated head is the head at which the specifications require that the turbine operating at full gate shall produce sufficient power to deliver the nameplate power output from the attached generator. Nameplate power ratings of the turbine and generator are based on this head, hence the term rated head.

When possible, it is preferable to have the rated head occur when the reservoir is at the weighted average water level. This rated head should equal the design head of the turbine to assure maximum overall plant efficiency. However, when the range of water levels is near the maximum, it may be necessary to have the design head deviate from the rated head so that the 125 - 65 percent limitation of design head will encompass the expected head range resulting from reservoir water level variations.

Powerplant size

The prime requirement of a multiple purpose project powerplant is to produce the maximum feasible amount of power compatible with other operating requirements of the project, from the scheduled discharge which is required to meet normal downstream demands. Therefore, the total installation of units, consisting of turbines, and their attached generators, should be capable of utilizing all of that scheduled discharge other than spillway releases, under all conditions of reservoir water levels. The characteristic performance curves of these units as illustrated in Figure 3 show the effect of the attached generator when the rated head is made equal to the design head of the turbine. Here the full gate discharge at minimum head approximately equals the discharge required to produce full generator output at maximum head. Both of these are about 70 percent of the maximum discharge capacity of the rated head. This provides a balanced design with



TYPICAL HYDROELECTRIC UNIT PERFORMANCE
(FRANCIS TYPE TURBINES)
FIGURE 3

minimum variation of discharge over the head range, and results in the minimum size for the turbine.

The rated nameplate power capacity of a plant operating over the maximum permissible range of head will in many cases be found to be approximately twice the base load capacity which is available 100 percent of the time. This base load capacity would be equivalent to the capacity at the minimum head. During a runoff period of about 25 years duration, the full gate capacity corresponding to this base load would occur in many projects only during a few months of the total period. This infrequent occurrence of solely base load suggests the desirability of contracting for sale of capacity and energy in excess of the base load. When the range of water levels is less than the maximum permitted by the turbine, the same conditions noted above exist, but to a proportionally less degree.

Number of Units

Limitations

Generally, it is preferable to have all units in a plant of like size and as large as possible, mainly to simplify maintenance, improve accessibility, and reduce the inventory of spare parts. However, the actual number of units into which a given plant is to be divided must be determined by consideration of the several limitations in physical size and mechanical performance.

Cost

In well-engineered hydroelectric plants, the overall cost per horsepower varies little with the number of units into which the plant is divided. The higher per horsepower cost of the smaller units is offset by the lower cost of the smaller service equipment and less expensive excavations. Since the spiral case and draft tube of the turbine set the basic overall dimensions of a unit bay, the maximum flow to be passed is the major factor determining the cost of the plant. Once the required flow and weighted average head has been established, varying the rated capacity of the generator attached to a given turbine will have less than 1 percent effect on the overall cost of the plant.

Sizes Available

A one-piece cast steel Francis runner with an outside diameter of about 17 feet, which would be capable of passing from 3,000 cfs for high head to 6,000 cfs for low head units, is the largest that can be shipped by rail. Segmented runners with field assembly are not used by the Bureau of Reclamation as they may have their strength impared due to vibration, and may loosen during the many years of service. Units with segmented runners are much heavier and more expensive than those with single piece runners due to the extra shop work required and to the difficulties of field assembly. Thus the maximum economy is achieved by the maximum unit size using a single piece runner. Cast stainless steel runners used in high head plants are presently limited, because of stainless steel casting difficulties, to approximately 6-1/2 feet diameter, passing approximately 600 cfs. However, built-up runners of welded construction properly stress relieved before final machining would be restricted only to transportation limitations.

Vertical shaft units of less than 3-foot discharge diameter and passing less than 200 cfs, require special care both in the machine design and the structure layout to provide the necessary accessibility for normal maintenance and repair. The resultant small water passages through the runner require closely spaced trashracks which in turn, may necessitate continuous mechanical raking to insure uninterrupted production of power.

Power Demand

Present day large transmission systems are usually supplied by several powerplants. Consequently, an entire plant could be shut down for properly scheduled maintenance and repair with little disturbance to the system. Isolated plants which are the major source of power for the system and which will be operating under a maximum range of head can, with two or more units installed, maintain their contract base load output with one unit out of service during high head periods. This is assuming that the base load output is established by available capacity at minimum head which may be as little as 50 percent of the capacity available at the weighted average head. This variation of capacity with head, coupled with a 95 percent availability factor of all hydro units of record, make it usually undesirable to install additional units for insurance of continuous power generation.

Part Gate Operations

Operation at small gate openings results in low efficiency, excessive vibration and accelerated damage from cavitation. It is preferable to restrict continuous extended operation to the range producing 80-percent efficiency or better as illustrated in Figure 4. This minimum gate will vary from 25 percent for units designed for 250 feet of head or higher to 60 percent for those designed for 100 feet of head. Thus, above 250 feet of head a 2 or 3 unit plant will be capable of handling a wide range of load, and all units can be of the same size, while below that head a 3 or 4 unit plant would be more suitable.

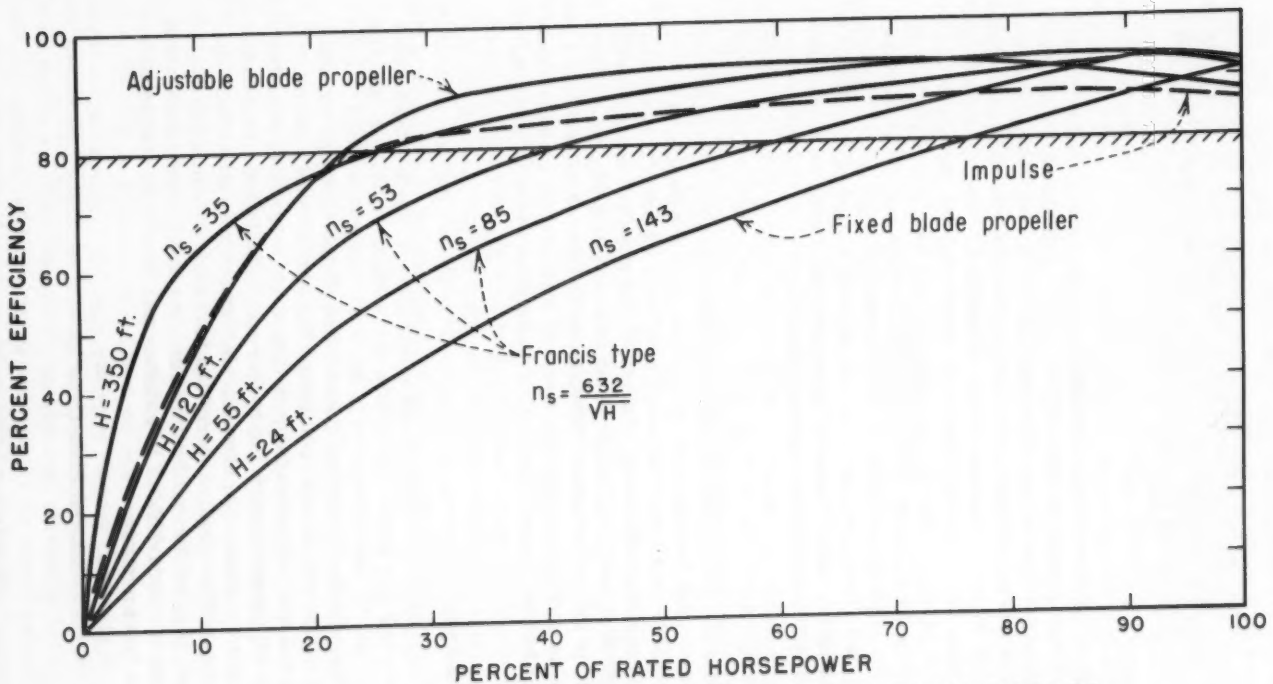
Maintenance and Repair

The service equipment of a powerplant is designed to handle maintenance and repair of one unit at a time. The crane, oil handling equipment, and other services, as well as the space required to set down component parts of a disassembled unit are all directly proportional to the physical size of the unit. A well balanced layout with service equipment that can be easily and efficiently handled, and with sufficient floor or deck area adequate to set down component parts will result from a two to four unit installation. This same complex of service equipment can economically handle up to ten units when the runner size limitations require more than four units to handle the discharge. However, it is seldom profitable to divide a small plant into several units as the small units are difficult to maintain and repair, as discussed in the paragraph on sizes available.

Rated Plant Capacity

Generator Rating

Most central powerplant prime movers, such as steam turbines, have a fixed maximum capacity just sufficient to drive their attached generator at



HYDRAULIC TURBINE CHARACTERISTIC EFFICIENCY CURVES AT DESIGN HEAD

FIGURE 4

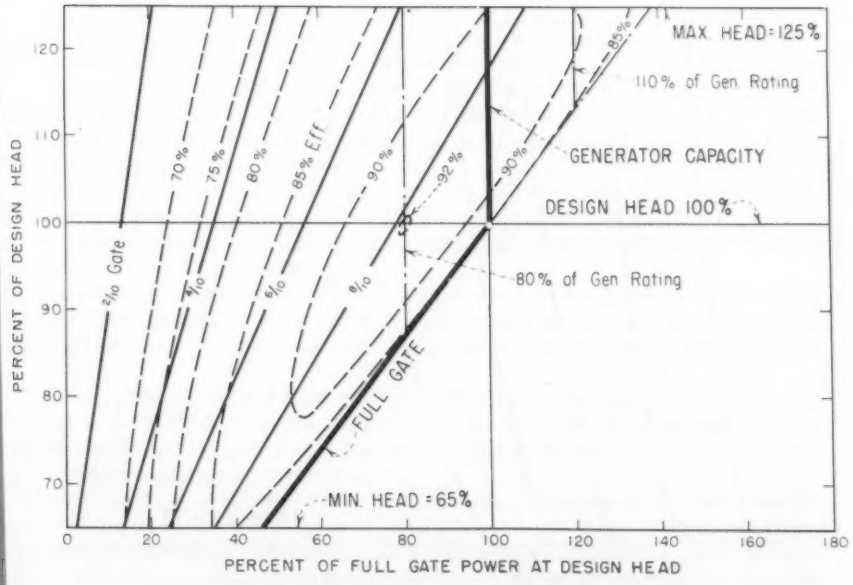
PERCENT OF DESIGN HEAD

full rated load. Francis type hydraulic prime movers, however, when operating over their maximum permissible range of head, have a full gate power capacity under the maximum permissible head, which is more than three times the power that could be produced under the minimum permissible head. The size of the turbine is established by the flow, and the size of the attached generator must be established primarily by the turbine capability rather than networks requirements.

The preferred generator size for a given turbine is that capacity at unity power factor which will just absorb full turbine output at the best efficiency or design head. This selection is supported by several factors based on turbine characteristics and project requirements.

Turbine Output

The typical efficiency contour curve of a Francis turbine and generator of Figure 5, shows unit power output under a variable head. Matching the generator to the turbine at the best efficiency head will permit operation over a large part of the best efficiency range and will result in more kilowatt-hours output for a given quantity of water. In addition, at the higher heads the unit can operate at better than 70-percent gate without exceeding generator rating, thus reducing the amount of maintenance and repair, and thereby increasing the net revenue. Matching the machines at materially less than design head



POWER OUTPUT CURVE
FRANCIS TURBINE
FIGURE 5

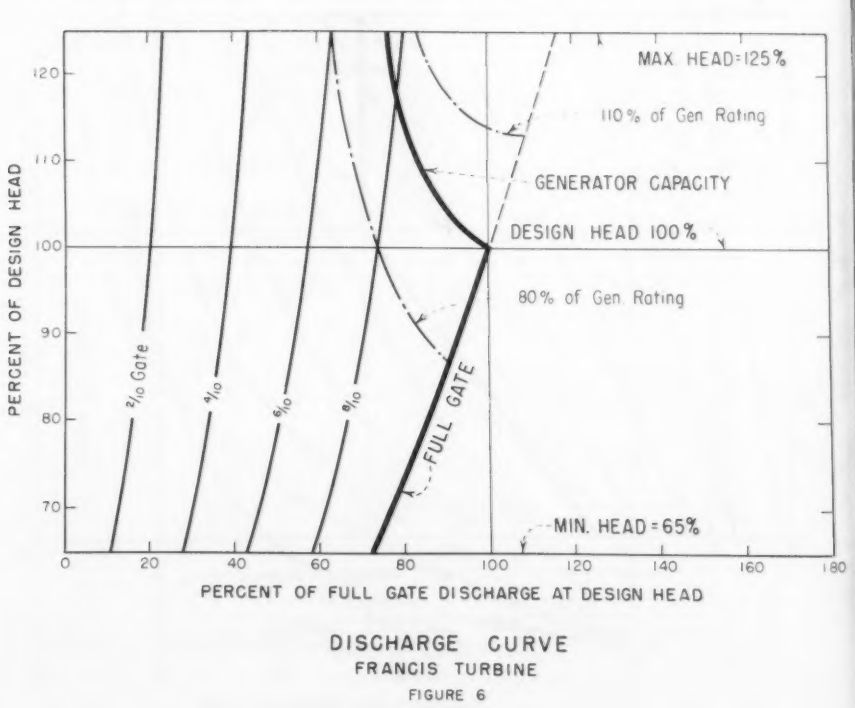
will result in a slight saving of initial cost, but will reduce the time that the unit can operate at good efficiency. Also, at the maximum head, because of the very small gate at which full generator output is achieved, it will be difficult to hold uniform speed under load variations or while synchronizing. Under special conditions, such as in systems with excessive load peaks, a matching at more than the design head would provide extra capacity during periods of high stream flow and full reservoir.

Discharge Capacity

The discharge curve of the same typical unit is shown on Figure 6, and illustrates another factor in matching the machines at the design head. The discharge capacity at minimum head, which is usually a basic requirement, is approximately equal to the discharge required to produce full generator capacity at maximum head. When matched at a lower head this high head capacity of discharge is less, and therefore, a larger turbine and generator will be required to fully utilize the total minimum flow.

Value of Power

The total kilowatt-hours produced is actually little affected by the size of the generator except for the effect of efficiency. The total kilowatt hours are produced by the passage of a given amount of water through the turbine, and



DISCHARGE CURVE
FRANCIS TURBINE
FIGURE 6

restrictions due to generator capacity only increase the time required to produce those kilowatt-hours.

Establishing the generator size and thereby the rated plant capacity through economic power studies using the primary concept of base load and surplus power may lead to erroneous findings not in accord with actual experience. This so-called surplus power is usually available a high percentage of the time and is valuable in meeting system peaks and in supplying interruptable power. In some cases, this surplus has become nearly as valuable as the base load power such as during the last war. During this period, Grand Coulee Powerplant produced a plant factor of over a hundred percent in one year due to continuous operation in excess of rated generator capacity for all units. Present day generators are rated much closer to their ultimate capacity and should seldom be operated in excess of their rating for extended periods of time.

SUMMARY

Power is the product of the storage and distribution of water in the western multipurpose projects of the Bureau of Reclamation. The revenue from that power is often a major source of income to the project and, as such, should be developed to its maximum feasible extent where economically justified without detriment to other purposes of the project. Also, reservoir water level variation should be held within the limits imposed by hydraulic turbine capability to assure a reasonable minimum amount of maintenance and repair and a stable unit under extreme conditions.

The physical size of the plant is dictated by the rate of discharge required to meet water demands. Since the discharge capability of the turbine increases with the head, the capability at minimum head is usually the basic factor which determines the plant size.

The rated head on the plant will normally be established by the weighted average water level of the reservoir. The design, or best efficiency head of the turbine should be as close to this rated head as the permissible range of head on the turbine will permit.

The rated kilowatt output capacity of the plant is established by the rated head and the turbine horsepower available at that head. This horsepower is established from the water demands with the turbine operating at the minimum head.

These criteria, when superimposed on the normal reservoir operational studies of irrigation projects, will produce an efficient plant whose output can be readily absorbed in a large power system without material detriment to the prime purpose of irrigation.

Power capacity over and above the base load capacity at the minimum head can be used for a high proportion of the time to meet system peaks and to supply interruptable demands. As such, it may command a price almost equal to the base load value and will provide a welcome addition to the plant revenue.

ACKNOWLEDGMENT

The procedure and conclusions outlined in this paper are drawn from reports of powerplant-operating personnel, field-conducted index tests, and turbine acceptance tests made by the Bureau of Reclamation on more than 40 turbine installations placed in operation in the last 25 years. Turbine acceptance tests were performed under the direction of Mr. I. A. Winter, Chief of the Hydraulic Machinery Branch of the Commissioner's office, Bureau of Reclamation, Denver, Colorado.

REFERENCE

1. "Selecting Hydraulic Reaction Turbines," ENGINEERING MONOGRAPH No. 20, Bureau of Reclamation, Denver Federal Center, Denver 25, Colorado.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

OCEAN COOLING WATER SYSTEMS FOR TWO THERMAL PLANTS[a]

L. T. Mariner,[1] F. ASCE and
W. A. Hunsucker,[2] F. ASCE

SYNOPSIS

This paper compares varied problems associated with the design and construction of circulating water facilities for two coastal thermal plants to serve the City of Los Angeles. Widely differing locations pose diverse design problems of sanding, erosion, and navigational requirements. Construction dewatering problems are illustrated and discussed.

INTRODUCTION

The Power System of the Los Angeles Department of Water and Power serves an incorporated area of about 450 square miles inhabited by approximately 2,400,000 people. Figure 1 shows the System in relation to the service area. The load on the System approximately doubles each ten years. Peak load reached a high of 1,435 MW in 1958. Available generating capability, on a two-hour loading basis, is 1,840 MW of which 1,170 MW is now provided by four steam plants. The System's hydro generating capability of approximately 670 MW includes firm power generated at Hoover Dam by the City from units belonging to the United States, Department of the Interior. The thermal generating facilities consist of: Harbor Steam Plant with five small

Note: Discussion open until January 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2133 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. PO 4, August, 1959.

a. Presented at convention of ASCE, Los Angeles, Calif., February 1959, under title: "Ocean Cooling Water Systems for the Scattergood and Haynes Plants, and Related Construction Dewatering Problems."

1. Principal Civ. Engr., Power System, Dept. of Water and Power, Los Angeles, Calif.
2. Sr. Structural Engr., Power System, Dept. of Water and Power, Los Angeles, Calif.

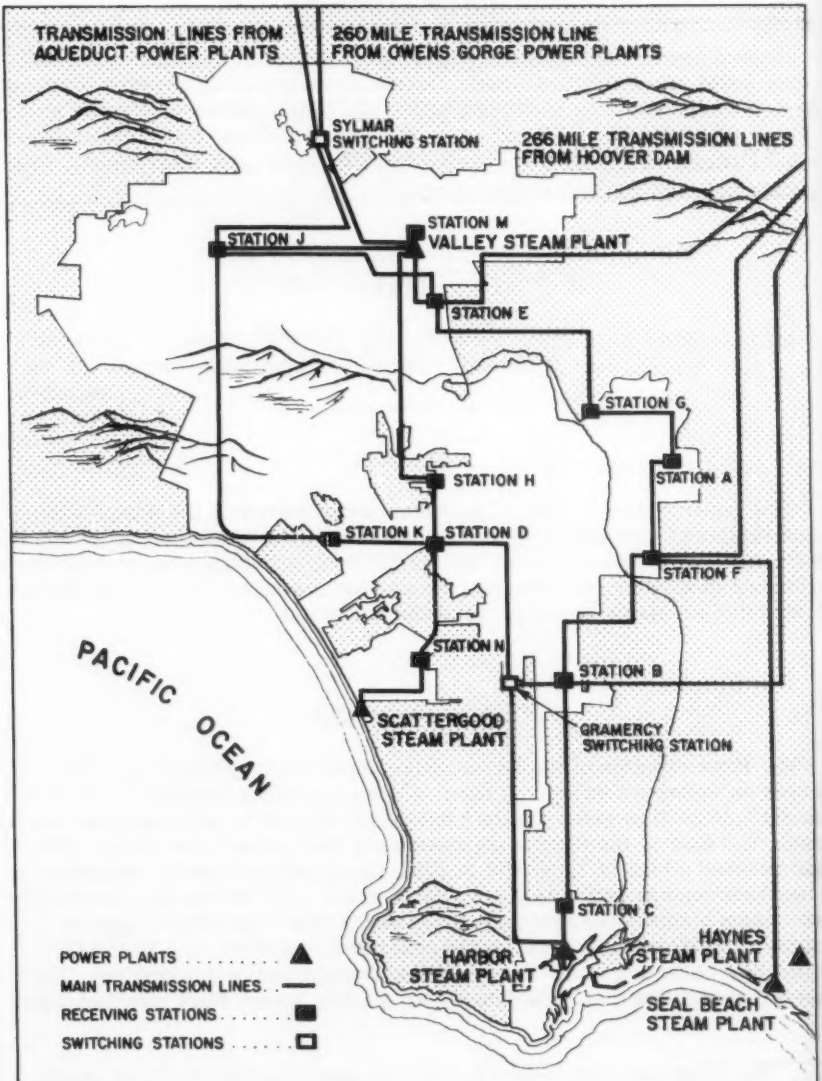

TRANSMISSION LINES FROM AQUEDUCT POWER PLANTS
260 MILE TRANSMISSION LINE FROM OWENS GORGE POWER PLANTS
SYLMAR SWITCHING STATION
266 MILE TRANSMISSION LINES FROM HOOVER DAM
STATION J
STATION M
VALLEY STEAM PLANT
STATION E
STATION G
STATION A
STATION H
STATION K
STATION D
STATION F
STATION N
STATION B
PACIFIC OCEAN
SCATTERGOOD STEAM PLANT
GRAMERCY SWITCHING STATION
STATION C
HAYNES STEAM PLANT
HARBOR STEAM PLANT
SEAL BEACH STEAM PLANT
POWER PLANTS
MAIN TRANSMISSION LINES
RECEIVING STATIONS
SWITCHING STATIONS
CALIFORNIA
Los Angeles
POWER SUPPLY SYSTEM
of the
LOS ANGELES DEPARTMENT of WATER and POWER

Figure 1

units totaling 355 MW; Valley Steam Plant having four units with a total capacity of 520 MW; one new 156 MW unit now in production at the Scattergood Steam Plant; and on standby, two units rated at 75 MW at Seal Beach, the oldest steam plant in the System.

To meet future load, the Department has two thermal plants under construction. Both will use sea water for cooling. Unit No. 2 of the Scattergood Plant on Santa Monica Bay is approaching completion, and will be on the line in August or September. The Haynes Plant, located a mile inland from Alamitos Bay, is in the initial stages of construction.

Scattergood Steam Plant

The site of the Scattergood Plant as it looked before construction is shown by Figure 2. It will be noted that the site is filled with wind-blown dunes of sand. The higher dunes were more than 140 feet above the road paralleling the ocean front and were about 175 feet above mean sea level. Figure 3 shows the site after approximately three million yards of sand were removed and used to widen the beach north of the site.

The layout of the plant, showing the completed unit, the unit under construction and future units, is shown by Figure 4. The circulating water system, seaward from and including the screen and pump chamber, is designed for cooling the first four units. Although space is provided for a six-unit plant, it is estimated that there will be major changes in the design of thermal generating units before the last two units are required at Scattergood. Therefore, circulating water facilities for the last two units have not been constructed although space has been provided.

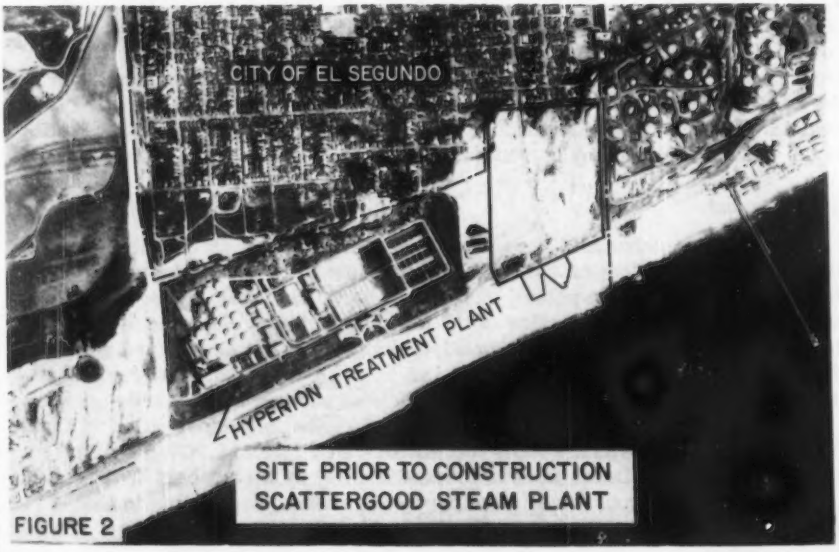


FIGURE 2

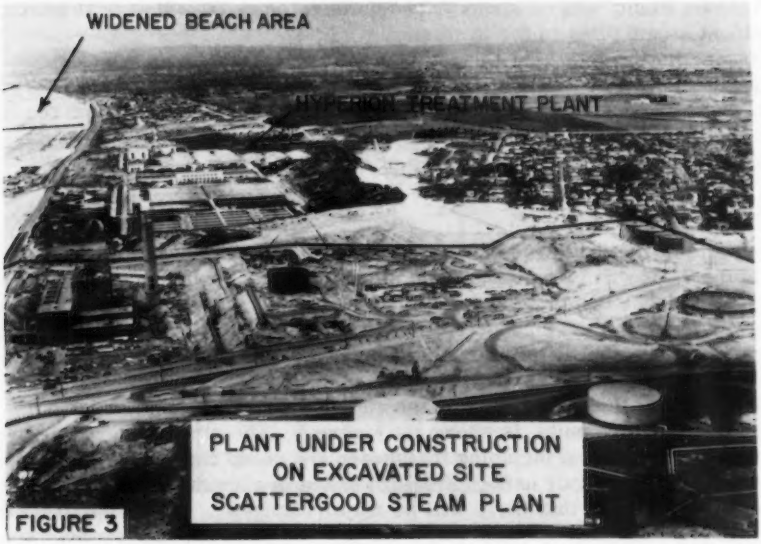


FIGURE 3

Circulating Water System

The characteristics of the circulating water system with respect to head, volume and velocity of flow, friction losses and general requirements, including flow reversal, were developed by Mechanical Design Engineers in considering condenser design criteria and operating problems resulting from uncontrolled growth of marine life. An extensive study was made of the hydraulic and thermal transients during flow reversal in the seaward tunnels. This study was conducted in collaboration with the Computer Staff of the School of Engineering of the University of California at Los Angeles.

Arrangement

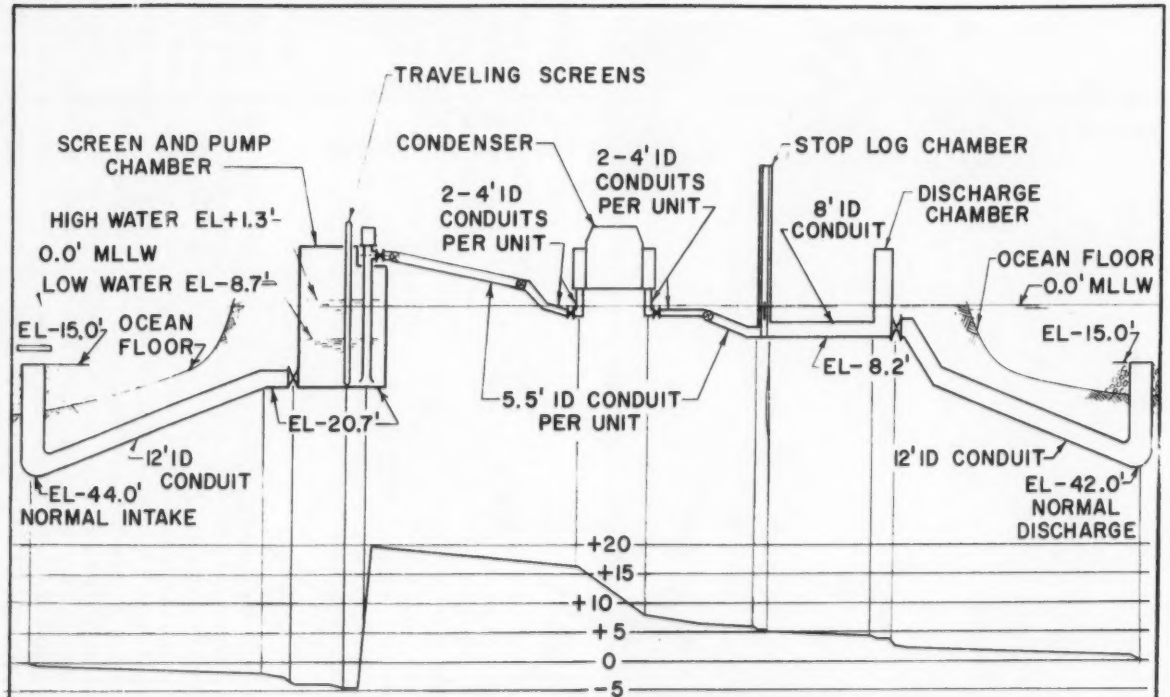
Figure 5 is a schematic representation of the overall circulating water system in vertical section. It indicates the flow from the intake structure, through the submarine pipeline, screen and pump chamber, to the condenser, back through a manifold section of the screen and pump chamber, and out to the discharge structure. In the manifold sections of the screen and pump chamber, four large butterfly valves, indicated in Figure 6, are provided to partially recirculate and to reverse the flow so as to control marine growth.

The control of marine growth was well explained in a paper titled "OCEAN COOLING WATER SYSTEM FOR 800 MEGAWATT STEAM POWER STATION"* (Huntington Beach in Southern California) by Mr. Robert Weight A.M. ASCE. Operating experience in the control of marine organisms in the coastal water of this area apparently leads to quite uniform practices which

*Presented at ASCE Convention, New York, N. Y., October 1958. (Proceedings Paper No. 1889)

OUTLET
INLET
N
PACIFIC OCEAN
LEGEND
UNIT 1 COMPLETED
UNIT 2 UNDER CONSTRUCTION
UNITS 3 & 4
FUTURE UNITS
SWITCH YARD & RECEIVING STATION AREA
FUEL OIL UNLOADING AREA
SCATTERGOOD STEAM PLANT
PLANT LAYOUT

FIGURE 4



VERTICAL LONGITUDINAL SECTION AND HYDRAULIC GRADIENT
CIRCULATING WATER SYSTEM
SCATTERGOOD STEAM PLANT

FIGURE 5

CIRCULATING WATER SYSTEM
SCATTERGOOD STEAM PLANT

FIGURE 5

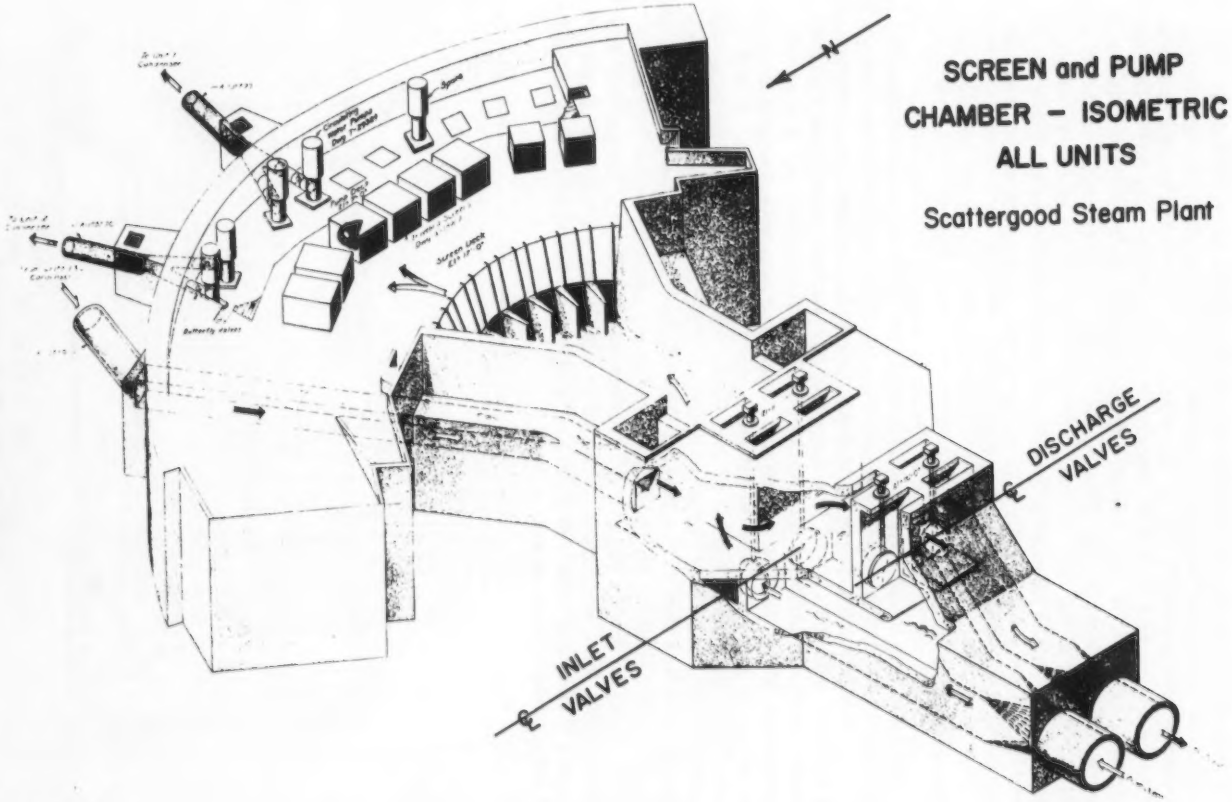


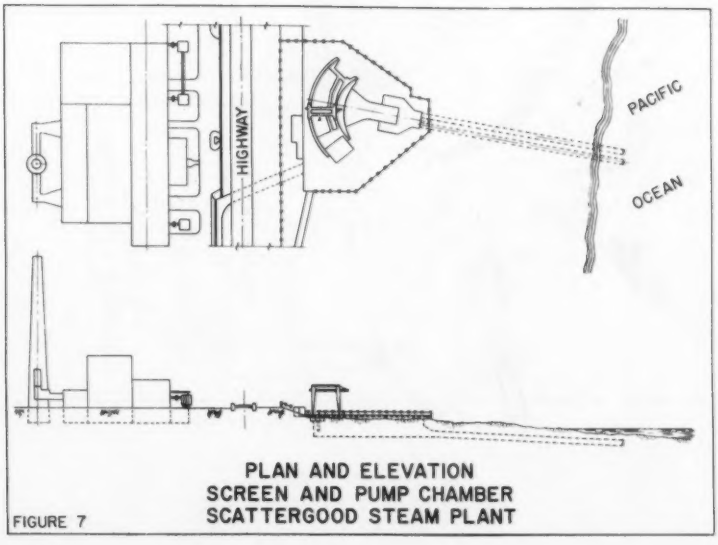
FIGURE 6

will be restated here by way of explanation of the design of the circulating water system.

Figure 6 is an isometric view of the screen and pump chamber. The normal flow pattern is indicated by arrows. Approximately monthly during the summer and bimonthly during the winter, the flow in the intake line is reversed and slowed so that a temperature of approximately 105° Fahrenheit is maintained for a period of from one and one-half to four hours. To obtain such temperatures in the effluent, the cooling water is recirculated by suitable operation of the four valves. The hot water is moved slowly seaward through the intake pipeline at a velocity controlled by the manifold valves which insures a sufficiently high temperature on the pipe walls to kill the mollusks and associated marine organisms.

Screen and Pump Chamber

For reasons of efficiency and economy, the screen and pump chamber was located on the beach side of the highway. However, this section of beach was a part of Dockweiler Beach State Park, devoted to recreational purposes under a contract with the City of Los Angeles. The surface area from which the public would be excluded was limited by State requirements. A compact, integrated design for the screen and pump chamber, with the deck at grade, as shown in Figure 7, was developed to meet this requirement. It will be noted that, with the exception of the crane, the structures are all essentially underground, located as close to the highway fill as was practicable. The fenced area was kept at a minimum with consideration given to later enclosing the space planned for the screen and pump chamber which will serve Units 5 and 6. In this regard, it is of interest to note that the project was responsible for a substantial increase in public beach area. As shown in Figure 3, the sand removed from the plant site widened more than two miles of public beach in the area.



PLAN AND ELEVATION
SCREEN AND PUMP CHAMBER
SCATTERGOOD STEAM PLANT

FIGURE 7

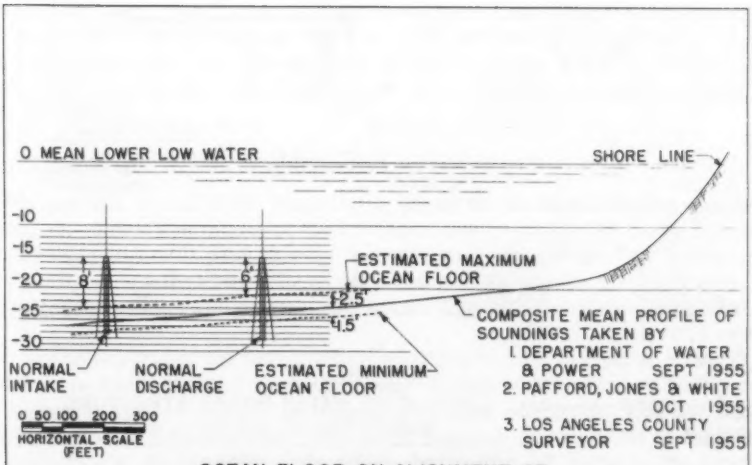
Location of Submarine Facilities

A number of factors were taken into consideration in determining the location of the submarine intake and discharge facilities for the circulating water. Investigations were made of the ocean currents, temperatures, and the physical conditions of the shore line and ocean floor in the vicinity of the plant site. It was found that the Santa Monica Bay is remarkably free from storms and the weather is generally favorable. This was substantiated by material published as far back as 1896 when the Federal Government investigated Santa Monica Bay as an alternate for the Los Angeles Harbor at San Pedro.

Previous oceanographic studies by other agencies were reviewed. It was found that, although the ocean floor intermittently varied a few feet in elevation over a period of several years, the average depth remained relatively constant. A vertical section (Figure 8) along the alignment of the proposed pipelines indicates the estimated variation in bottom elevation. The economic significance of this variation, in relation to the slope of the bottom, is more apparent when considered in connection with the criteria established for the location of the intake and discharge structures.

Intake and Discharge Structures

Navigational requirements indicated that it would be economically desirable to have the tops of the structures submerged approximately 15 feet below low tide or at elevation -15 Mean Lower Low Water. In addition, studies of wave characteristics in the bay also indicated a minimum submergence of 15 feet to avoid the heavy force of breaking waves and the hazards of entrained air. Also serious consideration was given to preventing the inflow of large quantities of suspended sand during periods of high wave action.



OCEAN FLOOR ON ALIGNMENT OF SUBMARINE CIRCULATING WATER CONDUITS SCATTERGOOD STEAM PLANT

FIGURE 8

Our consulting engineers* had in the past found that there was considerable sand in suspension, during adverse conditions, in a zone of from 8 to 10 feet in depth above the ocean floor. Usually it would be considered desirable to set the minimum elevation from 10 to 15 feet above the floor to provide a margin of safety. However, here the inclination of the ocean floor is slight. If Figure 8 had been presented at a natural scale the very gradual slope of the ocean floor would be apparent. It follows that the lengths of submarine pipeline required to get added depth would have been excessive. It was therefore decided to locate the intake riser in 25 feet of water and the discharge riser in approximately 23 feet of water, measured at low tide. It was believed that even during storms, there would be little tendency to deposit sand in either riser while discharging. While it is true that the normal discharge structure becomes the intake during the "boil out" process, the times for these occasions may be chosen when there is calm weather and little disturbance to the ocean floor sands.

Fish Repellant Measures at Intake

During the construction of the circulating water system for Scattergood Steam Plant, engineering studies of the fish problems encountered at the Southern California Edison Company's Redondo and El Segundo Plants in the Santa Monica Bay were being concluded. As a result of this research and the successful use of a large, disk-shaped, concrete slab termed a "velocity cap" at the El Segundo Plant, it was decided to install a similar slab above the Scattergood intake riser. The riser, with velocity cap, is shown in Figure 9. The velocity cap has the effect of producing a horizontal

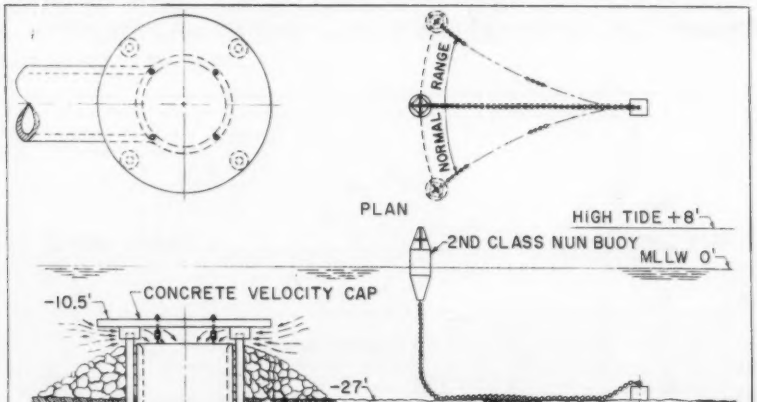


FIGURE 9

*Major Charles T. Leeds M. ASCE and Omar J. Lillevang A.M. ASCE have prepared comprehensive reports on wave action and sand movements in Santa Monica Bay.

accelerating velocity of approach in place of a vertical velocity of approach. Mr. Weight's paper on this subject reports that ocean fish are accustomed to drift with vertical movement but apparently oppose horizontal accelerating movement. It is reported that the device is a highly effective fish barrier.

Intake Obstruction Marker

The installation of the velocity cap reduced the clearance between the intake structure and the water surface at low tide from 15 feet to 10.5 feet, measured from MLLW Datum. This reduced clearance in navigable water was not considered adequate. An obstruction marker buoy was therefore installed to caution all vessels to keep to seaward. It is believed that the modified nun buoy was the first one installed for this purpose in this area. It was designed by the United States Coast Guard with a built-in radar reflector* as shown by Figure 10.

Consideration was given to the likelihood that seagulls might not feel full responsibility to perpetuate the red and black color bands which distinguish an obstruction marker of this type. The large, delicate coils of stainless steel, which are on every flat surface, have been effective in keeping the birds from utilizing the buoy for their purposes.

Haynes Steam Plant

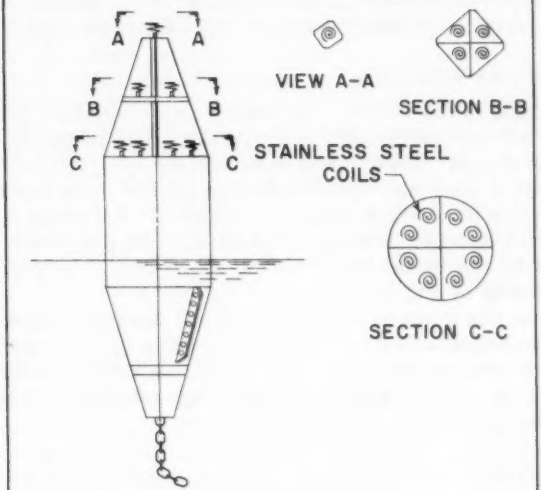
Conditions at the Haynes Plant Site differ in many ways from those encountered at Santa Monica Bay. An air photo (Figure 11) taken in the winter of 1938 shows the site under adverse circumstances, completely inundated. In the foreground may be seen the Department's smallest and oldest thermal plant, the 75 MW Seal Beach Plant.

Figure 12 is a recent photograph of the site with a perspective view of the Haynes Plant superimposed. The site has been filled and raised in elevation approximately three feet, and extensive drainage improvements have been made in the area. The levees of the flood control channel have been raised. For local flood control a large settling basin and pumping plant have been constructed to drain the area east and northeast of the San Gabriel River Flood Control Channel. The Department is now confident that the site is safe from flooding.

Circulating Water System Arrangement

The general layout of the circulating water facilities at Haynes is shown in Figure 13. From the intake at the bulkhead line of the Long Beach Marina, the cooling water will pass through substructures under the San Gabriel River and Pacific Coast Highway into an open channel leading to the plant site. Integrated screening and pumping facilities for the ultimate development are

*The upper portion of the buoy, which would normally consist of a conic section, is made of two intersecting steel plates which act as a "corner reflector." This size reflector functions rather efficiently with the radar wave lengths in common use. Any radar impulse striking one of the plates is reflected to the opposite plate which, being at a 90° angle to the first plate, reflects the impulse directly back to the source. This reflected impulse is relatively concentrated and results in a well defined "pip" on the radarscope.



2ND CLASS NUN BUOY WITH RADAR REFLECTOR
OBSTRUCTION MARKER
CIRCULATING WATER INTAKE STRUCTURE
SCATTERGOOD STEAM PLANT

FIGURE 10

not contemplated for the Haynes Plant. Each unit or pair of units will have a separate installation for this purpose. Discharge from the plant is to be by substructures opening into the flood control channel. The intake structure, closed conduits, and the open channel to the plant are designed to handle a maximum of approximately 2,000 cubic feet per second to serve the ultimate 6-unit plant.

Figure 14 is a schematic representation of a vertical section through the complete system. The relatively long hydraulic grade line from intake to discharge is not immediately apparent because the intake channel has been fore-shortened. Although the Haynes Plant Site is a considerable distance from the cooling water source, zoning and other restrictions affecting land usage have so limited the availability of sites for thermal plants that the Haynes Site has many advantages over available alternates.

Intake Facility

The City of Long Beach is currently building an addition to its Long Beach Marina as shown in air photo designated Figure 15. The circulating water intake for Haynes has been incorporated in the design of this addition to the Marina. It became evident that the proposed Marina as a source of cooling

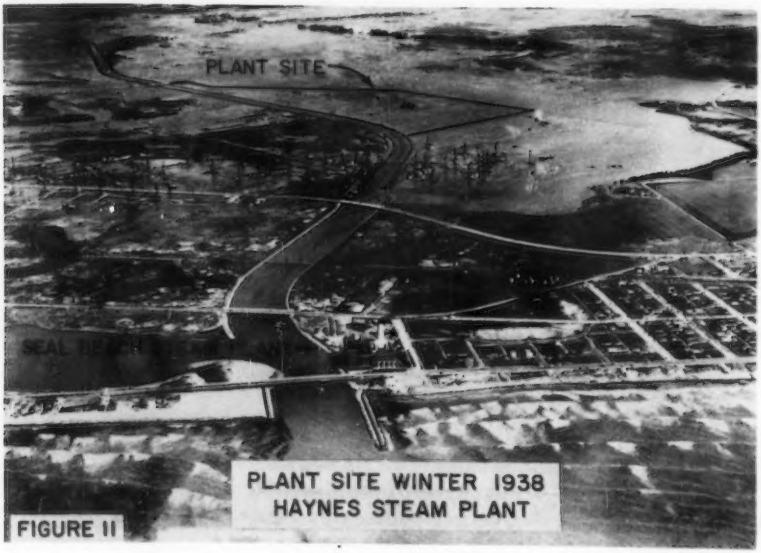
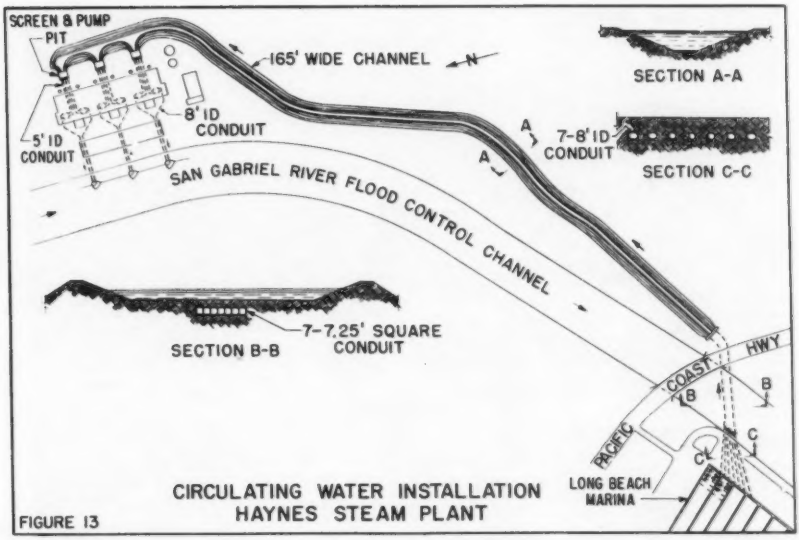
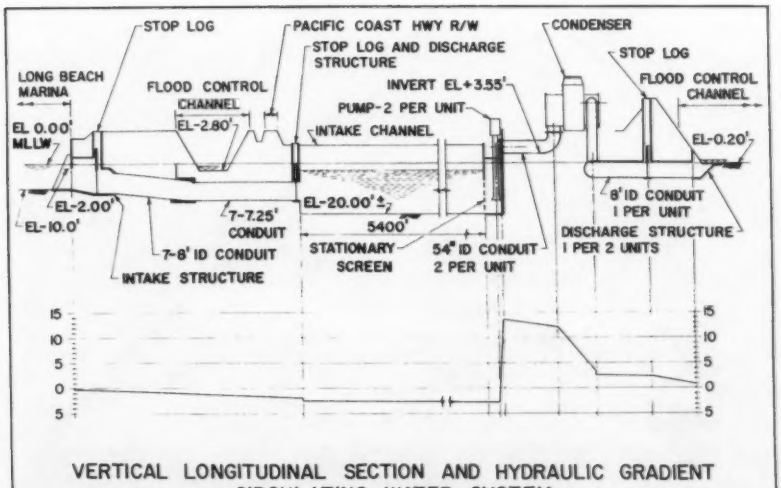


FIGURE 11

FIGURE 12



CIRCULATING WATER INSTALLATION
HAYNES STEAM PLANT
FIGURE 13



VERTICAL LONGITUDINAL SECTION AND HYDRAULIC GRADIENT
CIRCULATING WATER SYSTEM
HAYNES STEAM PLANT
FIGURE 14

water for Haynes would benefit both the City of Long Beach and the Los Angeles Department of Water and Power. As shown in Figure 16, the intake structure constitutes one bulkhead of the new Marina. It appeared most economical to combine both projects in one contract, resulting in a joint venture by both cities and the award of one construction contract.

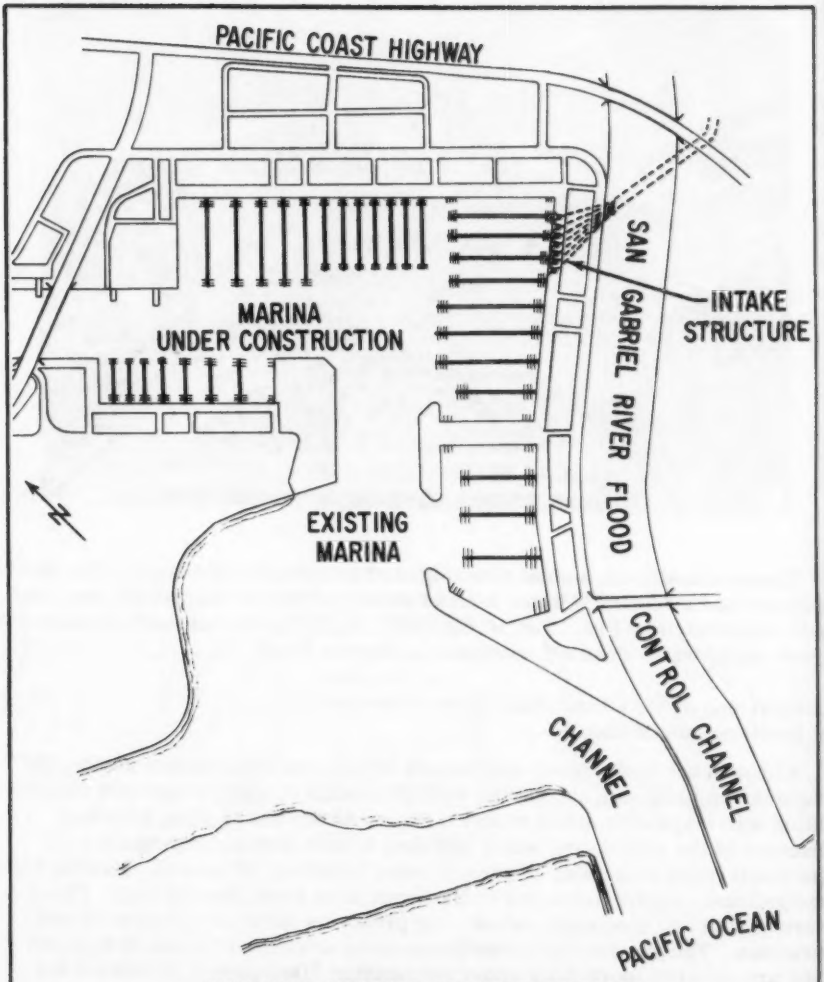
FIGURE 15

There will also be mutual advantages of an operational nature. The Marina will provide a source of water without entrained air or suspended sand, and with relatively few fish. The Marina will benefit by the constant movement of fresh ocean water drawn through to the Haynes Plant.

Comparison of the Circulating Water Systems at Scattergood and Haynes

Although the Scattergood and Haynes Plants are both coastal plants, the engineering problems associated with the design of their respective circulating water systems differ widely. Figure 17 shows, in plan, principal features of the circulating water systems at both plants. The system for the Scattergood Plant was adapted to meet problems of erosion, sanding and navigational requirements due to its location on Santa Monica Bay. There were varied and continuing dewatering problems during the course of construction. The ground water conditions were considered in the design and they are covered more fully under the heading "Dewatering Problems for Screen and Pump Chamber - Scattergood Steam Plant." In contrast, the design of a circulating water system for the Haynes Plant presents interesting problems of a different nature since the plant will be located a mile inland from the Long Beach Marina at Alamitos Bay.

Although the circulating water system for Haynes will be much longer, there are offsetting advantages in location. It is estimated that the overall capital cost of the Haynes circulating water system, per megawatt of ultimate capacity, will compare favorably with the cost for such facilities at the Scattergood Plant. It is perhaps apparent that the friction losses in the trapezoidal channel at the Haynes Plant would be much less than friction losses in the submarine pipelines at Scattergood. The pumping head and



HAYNES STEAM PLANT INTAKE
AT
LONG BEACH MARINA

FIGURE 16

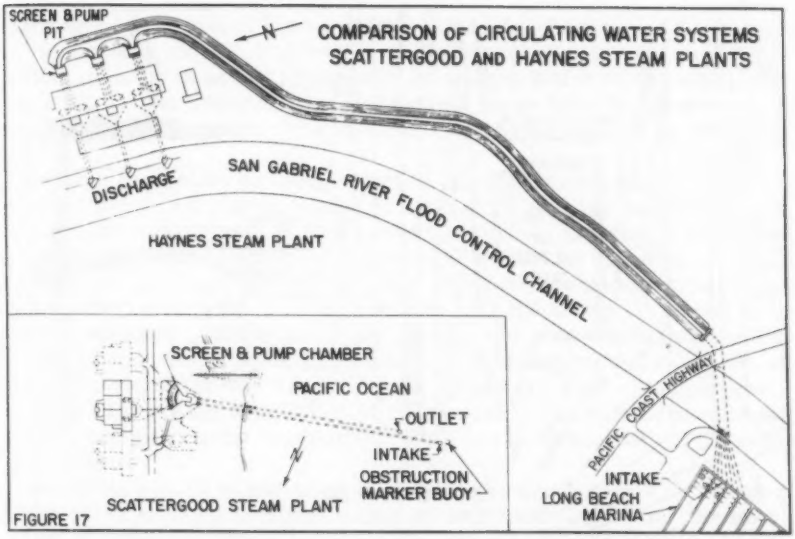


FIGURE 17

annual operating costs per megawatt for circulating water at Haynes are therefore expected to be somewhat lower than those for Scattergood.

If cost estimates prove to be correct, it is evident that a more remote site for a plant taking cooling water from an improved harbor may often provide a more economical location for a conventional steam plant than a site fronting on the ocean beach. A factor frequently either overlooked or improperly evaluated in cost estimating, is the reduction in cost of corrosion protection and plant maintenance associated with the more remote location.

Dewatering Problems for Screen and Pump Chamber - Scattergood Steam Plant

Figure 18 shows the site for the Scattergood Steam Plant which is on sandy soil with the water table at elevation +3.0 Mean Sea Level Datum. Excavation for the major portion of the steam plant and accessory structures extended below the water table and it was therefore necessary to unwater the site.

A system of well points was installed around the steam plant proper, and the area was unwatered without incident. Because of the fact that the screen and pump chamber is removed from the main building, a separate well point system was used to unwater the excavation for the screen and pump chamber.

The screen and pump chamber is located approximately 300 feet easterly of the shore line, and natural grade is +16.0. The bottom surface of the base mat of the structure is at elevation -27.5.

Well Point System

The contractor elected to unwater the excavation by means of a system of well points which was installed in two stages as indicated in Figure 19. The

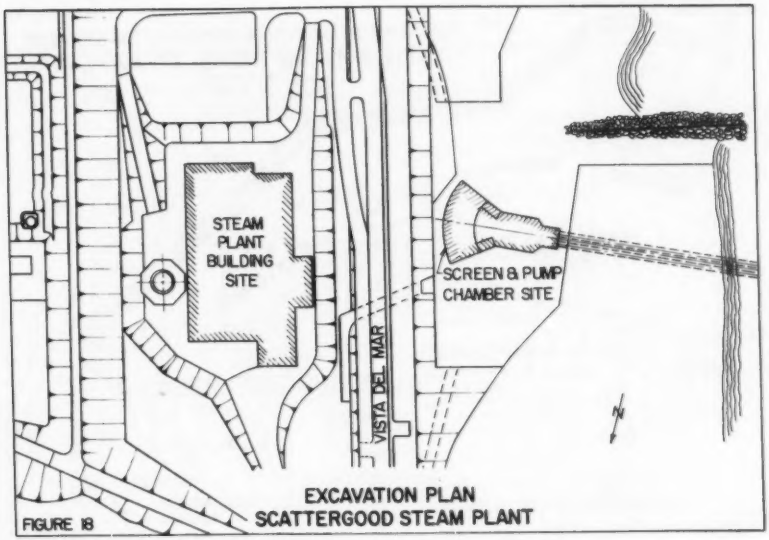


FIGURE 18 EXCAVATION PLAN SCATTERGOOD STEAM PLANT

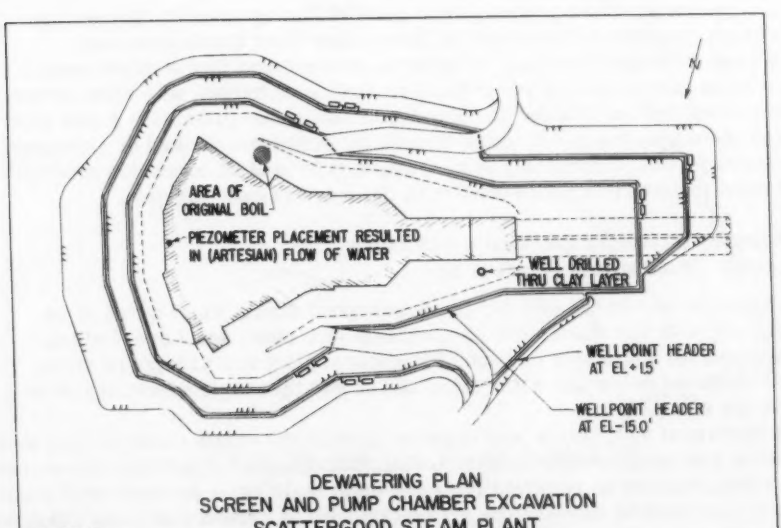

N
AREA OF ORIGINAL BOIL
PIEZOMETER PLACEMENT RESULTED IN (ARTESIAN) FLOW OF WATER
WELL DRILLED THRU CLAY LAYER
WELLPOINT HEADER AT EL +1.5'
WELLPOINT HEADER AT EL -15.0'

FIGURE 19 DEWATERING PLAN SCREEN AND PUMP CHAMBER EXCAVATION SCATTERGOOD STEAM PLANT

upper stage header was at elevation +1.5 and the lower stage header at elevation -15.0. In order that the soil under the foundation mat would not be disturbed, the specifications required that the water table be lowered in such manner that it would be maintained a minimum of 3 feet below the surface of the excavated area as the excavation proceeded.

Aquifer under Pressure

During the progress of the excavation it was determined that the underlying formations contained an aquifer under pressure. Peizometer tubes installed in the area and extending into the aquifer indicated the static pressure in the aquifer was sufficient to raise the water level to approximately elevation -13. This aquifer was below an impervious stratum at about elevation -45.

When the excavation had progressed to approximately elevation -23 in the easterly portion of the area, a sand boil appeared near the southeast corner of the proposed structure. It was feared that further excavation might produce additional boiling and disturbance to the foundation material on which the proposed structure would rest. In an effort to reduce the water pressure in the aquifer, a 30-inch well was installed near the northwest corner of the proposed structure. The initial flow from the well was approximately 700 gallons per minute. In order to obtain more data regarding the aquifer, an additional piezometer was installed near the easterly side of the proposed structure. During the installation of the piezometer an additional well was produced at this location which also acted so as to reduce the water pressure in the aquifer.

Both wells served to withdraw water from the aquifer for a period of approximately two months, either by flowing or by pumping. After the base slab and a portion of the walls for the structure were poured and a part of the backfill had been placed, the piezometer well near the easterly side of the structure was killed by pumping, into the well, a mixture of barium and oil well drilling mud. This procedure stopped the flow from the well.

Flood Water

After the excavation had been completed, and during the progress of installing forms for the walls of the structure, the work area was flooded by water released from outside the steam plant site as indicated in Figures 20 and 21. This incident was not related to the aquifer under the screen and pump structure. The resulting damage was extensive and the work was delayed approximately six weeks.

Backfill

After the flood water was removed from the area and the site was cleared up, construction and backfill operations proceeded as before. The unwatering system was maintained in operation as it was during the excavation so as to assist in compacting the backfill. Backfill material was principally sand and it was compacted by local flooding and vibrating. The underwatering system provided a means of removing the water used for flooding the backfill and thus caused a downward flow of water which was helpful in compacting the backfill.

Dewatering for Haynes Steam Plant

Preliminary investigation indicates that no serious problem should be encountered in unwatering the site for the Haynes Steam Plant. At this time, construction has not been started on the plant. The installation of the circulating water system will include a crossing under the San Gabriel River with seven waterways, each waterway to be 8 feet in diameter or equivalent cross

FIGURE 20 FLOOD DAMAGE SCREEN & PUMP CHAMBER SCATTERGOOD STEAM PLANT

FIGURE 21 FLOOD DAMAGE SCREEN & PUMP CHAMBER SCATTERGOOD STEAM PLANT

section. Section "B-B" of Figure 13 indicates the design of this undercrossing and the profile appears on Figure 14. This item of work may involve the most difficult water control problem on this project. A similar crossing was successfully installed, however, for the Seal Beach Steam Plant in 1943-1944 and it is expected that the problem will be no greater at this location.

ACKNOWLEDGMENT

Overall engineering design and construction of these two projects has been under the direction of E. L. Kanouse, Fellow, AIEE, Engineer of Design and Construction and B. G. Hume, M., ASME., Engineer in charge of Steam Plant Design. J. F. Mariscal, M., AIEE, is Engineer in charge of Construction, with W. C. Mason as Project Engineer.

At Scattergood, construction of circulating water facilities between the screen and pump chamber and the steam plant was performed by the Kemper Construction Company. The screen and pump chamber was constructed by the H. C. Smith Construction Company, and the Macco Corporation installed the submarine pipelines of reinforced concrete, which were manufactured by the United Concrete Pipe Corporation. The Department of Water and Power acts as General Contractor on the project as a whole.

The Guy F. Atkinson Company is now enlarging the Long Beach Marina. The Haynes Circulating Water Intake Structure is a part of that project.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION

Page

Underground Power Plants in Canada, by A. W. F. McQueen, C. N. Simpson and I. W. McCaig. (Proc. Paper 1670, June, 1958. Prior discussion: 1953. Discussion closed.)
by A. W. F. McQueen, C. N. Simpson and I. W. McCaig (closure) . 89

Rockfill Dams: Kenney and Cheakamus Dams, by William G. Huber. (Proc. Paper 1671 June, 1958. Prior discussion: none. Discussion closed.)
by J. Barry Cooke 93

Underground Power Plants in Scotland, by C. M. Roberts. (Proc. Paper 1675, June, 1958. Prior discussion: 1953. Discussion closed.)
by C. M. Roberts (closure). 97

Performance of Cogswell and San Gabriel Rockfill Dams, by Paul Baumann. (Proc. Paper 1687, June, 1958. Prior discussion: 1953. Discussion closed.)
by Paul Baumann (closure). 101

Rockfill Dams: The Bersimis Sloping Core Dams, by F. W. Patterson and D. H. MacDonald. (Proc. Paper 1740, August, 1958. Prior discussion: 1953. Discussion closed.)
by F. W. Patterson and D. H. MacDonald (closure). 109

Rockfill Dams: The Paradela Concrete Face Dam, by Luis Henrique Gomes Fernandes, Edgard de Oliveira, and Nuno de Vasconcelos Porto. (Proc. Paper 1747, August, 1958. Prior discussion: 2013. Discussion closed.)
by Armondo da Palma Carlos. 111

Rockfill Dams: Design of Cougar Central Core Dam, by Paul Thurber. (Proc. Paper 1749, August, 1958. Prior discussion: none. Discussion closed.)
by J. B. Cooke. 119

Note: Paper 2139 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, PO 4, August, 1959.

UNDERGROUND POWER PLANTS IN CANADA[a]

Closure by A. W. F. McQueen, C. N. Simpson and I. W. McCaig

A. W. F. McQUEEN,[1] F. ASCE, C. N. SIMPSON,[2] F. ASCE and I. W. McCAIG.[3]—In his discussion Mr. Lawton has correctly re-emphasized that a comprehensive geological investigation must be made before the proper lay-out of an underground power house and its associated water conduits can be determined.

The wisdom of undertaking such a comprehensive geological study was proven at the Chute-des-Passes project where information obtained from the geological investigations showed that it would be necessary to change the orientation and position of the power house and draft tube manifold quite radically from the position and orientation selected for preliminary studies of the development. This is shown graphically on the key plan of Fig. 12.

Geological investigations including surface mapping and diamond drilling in the power house area revealed that the country rock is a jointed by competent paragneiss and that the strike of the two major joint systems as well as the strike of the gneissosity is approximately north-south. If the preliminary power house orientation had been adopted without due regard for the geological conditions, the largest rock spans and the most difficult part of the excavation for the development would have been located such that the rock's arching action would not have been utilized to the greatest extent. Numerous rock falls could then have been expected.

The principal geological features in the vicinity of the power house as finally located are shown on the block diagram of Fig. 12. The diagram clearly indicates that proper orientation of the underground excavations minimized, to the greatest possible extent, difficulties which might have been encountered due to structural weakness of the rock. A small fault which cut across the north-east corner of the draft tube manifold presented only a local problem and was not difficult to control. No serious interruptions or delays were encountered during excavation of the power house and all major phases of construction were executed in accordance with the pre-arranged schedule.

Mr. Lawton also refers to the European practice of setting the penstocks at an angle to the axis of the power house. If rock conditions were generally poor this might indeed reduce the difficulties which could be encountered in excavating the penstocks parallel to a plane of weakness. At Chute-des-Passes it was unnecessary to adopt this procedure which incidentally presents some problems in the arrangement of equipment in the power house.

---

a. Proc. Paper 1670, June, 1958, by A. W. F. McQueen, C. N. Simpson and I. W. McCaig.

1. Pres., H. G. Acres & Co., Ltd., Niagara Falls, Canada.
2. Chf. Engr., Hydr. Div., H. G. Acres & Co., Ltd., Niagara Falls, Canada.
3. Hydr. Div., H. G. Acres & Co., Ltd., Niagara Falls, Canada.

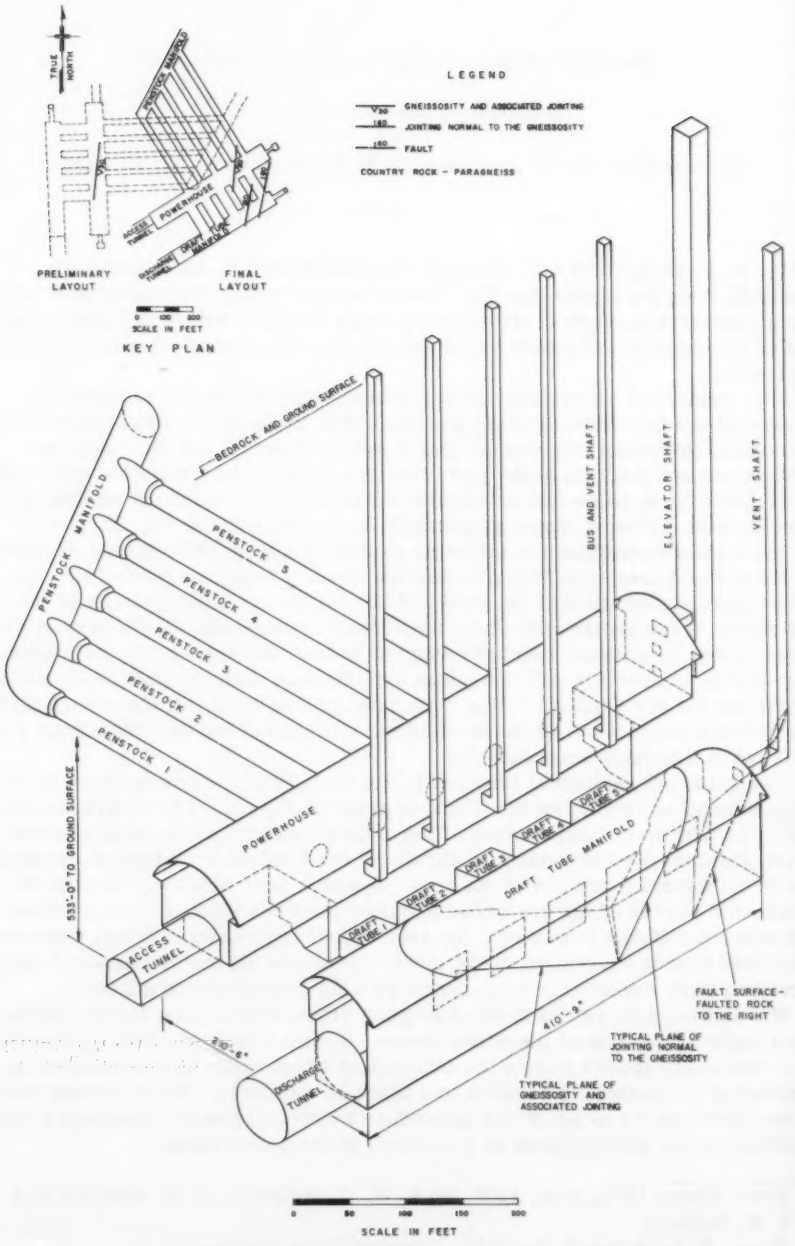


FIG. 12 BLOCK DIAGRAM SHOWING GEOLOGY IN AREA OF CHUTE-DES-PASSES POWERHOUSE

Instruments for measurement of rock movement were not installed within the Bersimis No. 1 and Chute-des-Passes power houses. If either power house had been excavated in an area where geological conditions were less stable than is the case in the paragneisses of the Canadian Shield, it would have been necessary to install instrumentation for such measurements.

ROCKFILL DAMS: KENNEY AND CHEAKAMUS DAMS[a]

Discussion by J. Barry Cooke

J. BARRY COOKE,[1] F. ASCE.—The author has provided valuable comment and data on two particularly notable rockfill dams. Kenney Dam was a substantial step from Nantahala Dam and has an important place in the successful development of the sloping core rockfill dam. Cheakamus, though not particularly high, is of substantial significance in its being constructed on glacial, alluvial and talus deposits without cutoff to bedrock. The Cheakamus Dam and the Bersimus Dam are examples of careful adaptation of rockfill dams to difficult sites. The proposed High Aswan Dam, to be constructed on a several hundred foot depth of sand and gravel, will further lead to increased use of rockfill dams at difficult sites.

The author states that the lateral movement (cross-valley component) of settlement points on Kenney Dam seems inconsistent with theory and says, ". . . The center point of a fill in a symmetrical valley should hold its original cross-valley position with reference to fixed points at the ends of the dam, during vertical and downstream displacements . . . At Kenney Dam both on the crest and along the berm, the total lateral movement to date approximates 6 inches at the center reducing to about 2 inches near the left abutment, and consistently towards that end of the dam . . ." These movements indicate that lateral compression has developed in the left half of the dam and lateral tension in the right half, rather than the expected tensions near both abutments and compression at the center. More detailed data would be of value. The writer agrees that the center of a symmetrical rockfill dam should not move laterally if the dam were symmetrically constructed. The lateral movement is basically from each abutment toward the center, but it is also related to the direction of dumping. The rockfill at Kenney Dam was constructed from the left to right bank, and that is believed to be the reason for the lateral movement being from the right to left bank. Lateral movement is greatest in the direction opposite to that of dumping the rockfill. Measurements on several dams indicate this and reasoning can be developed to make it appear logical.

The lateral settlement at Salt Springs Dam is greatest in the direction opposite to that of dumping the rockfill. The lower 170 feet of Salt Springs Dam was dumped from right to left abutment. The next 75 foot lift was also, except for a narrow face lift from the left abutment. The top lift was dumped from both abutments but its influence on lateral movements is not very great. Fig. 4, of Paper 1737 of this Symposium, shows that the compaction during dumping was greatest in the direction of the dumped slope, even when the settlement point was 100 to 150 feet back from the berm at which dumping was

a. Proc. Paper 1671, June, 1958, by William G. Huber.

1. Superv. Civ. Engr., Pacific Gas and Electric Co., San Francisco, Calif.

taking place. Points 3 - 5, Fig. 4, began to change direction of lateral movement as the fill berm progressed 200 to 300 feet beyond them. Fig. 8 of Paper 1737 shows the lateral movement to be predominantly to the right. On Line F all points moved to the right, in the opposite direction from dumping of the major portion of the fill. Lateral movements of the 245 foot high Lower Bear River Dam No. 1 were small but the upper left abutment joint openings, Fig. 17A, indicate greatest lateral movement being opposite to direction of dumping the fill. All the rockfill of Lower Bear River Dam No. 1 was constructed from the right abutment.

The Paradela concrete face rockfill dam, 368 feet high, was all constructed from the left to right abutment and lateral movement was predominantly from right to left. Vertical joints near the right abutment opened one to several inches, whereas the vertical joints near the left abutment did not move or closed about one-half inch. The design and construction of the dam is comprehensively presented in Paper 1747. The data on lateral movement at Paradela is given with the permission of Mr. Gomes Fernandes, Hidro Electrica do Cavado.

The lateral movements of the above three high concrete face rockfill dams appear to be substantially influenced by the direction of dumping the rockfill. Though Kenney Dam is a sloping core type, it is considered that its settlement should be similar to that of a concrete face rockfill dam, and the data presented by the author seems to confirm this for all components of the crest settlement. This similarity in crest settlement does not exist for the central core type of rockfill dam, for which all movements are less and are much less influenced by the rockfill.

It seems logical that lateral settlement should tend to be in the opposite direction to the direction of dumping. As dumping takes place the surface slides and impact of large rocks sliding down the slope tend to compact the fill in that direction. Also a substantial contact of the rocks in a line down the dumped slope is necessary to stop the rocks as they fall. The line of rock contacts is not as well established in a direction approximately normal to the dumped slope. When the differential vertical and downstream displacements take place the lateral readjustment would reasonably tend to be in the direction of least compaction; i.e., the direction opposite to the direction of dumping the rockfill. It appears that the magnitude of the effect of the direction of dumping on lateral movement is as great as the magnitude of the basic lateral settlement from both abutments to the center of a symmetrically constructed dam.

For dams of less than 325 feet in height these lateral movements are probably not of consequence. However, for higher dams it may be that consideration should be given to require a certain minimum amount of dumping from each abutment in order to minimize lateral tensions. Though the ideal design would require the dumped fills to close at the center, conditions are usually such that it is practical and economic to construct more dumped rockfill from one abutment than the other. Consequently, design restrictions on construction should be kept to a minimum. Since the lower lifts have the greatest effect on the lateral movement, the design requirements on the direction of dumping might be limited to them.

There is much remaining to be learned about the behavior of high rockfills and the adapting of rockfill dams to difficult sites. The publishing of construction data and detailed measurements on the existing dams will make it possible to design and construct higher rockfill dams with maximum safety

and economy. The cooperation of Mr. William G. Huber, The Aluminum Company of Canada and B. C. Electric Company in presenting data on Kenney and Cheakamus Dams has resulted in a valuable contribution to this Symposium.

UNDERGROUND POWER PLANTS IN SCOTLAND

Closure by C. M. Roberts

C. M. ROBERTS.[1]—The author wishes to thank the contributors for their remarks and would draw attention to Mr. P. L. Aitken's contribution which was printed on page 150 of Proc. Paper No. 1953.

Mr. Milne's contribution is a valuable addition to the paper in that it deals, in some considerable detail, with the construction problems encountered in the Glenmoriston Power Station.

Comparative estimates of underground and surface power station arrangements showed the former to be more expensive. The underground arrangement at Glenmoriston was chosen for various reasons including:

(a) Economy in cement and steel.
(b) Elimination of about 4 miles of 132 KV transmission line.
(c) Proximity to the future Livishie power station allowing for provision of common services.
(d) Anticipation of keen tunnel excavation rates which would favour the underground arrangement. (This had proved to be the case).

The economies effected by putting the Ceannacroc Station underground were largely due to the saving in time that resulted from the provision of a 16 MW machine similar to those in Glenmoriston. By avoiding the necessity for a long high pressure steel pipe there was a considerable saving of steel which, at the time, was in short supply.

With regard to the design for the arched roof of the Ceannacroc station it was decided that only a simplified approach was justified. A number of assumed loadings with wedge loadings between 45° and 60° included angle were investigated for the arch in addition to the condition of a rock load equivalent to 1/3 span of the arch (15 ft.).

A typical check loading was:

Rock: Active pressure from a wedge with 45° included angle.
Concrete: Dead load of arch.
Water: 10 ft. head distributed over full area.
Temperature range: ± 15° F.
Shrinkage equivalent to: - 15° F.

The strengthening of the rock above the concrete arch by grouting undoubtedly contributed to the strength of the roof. The same design assumptions were adopted for the roof of the Glenmoriston station. As commented on by Mr. Guthrie Brown there was a different treatment of the arch springings at

a. Proc. Paper 1675, June, 1958, by C. M. Roberts.
1. Partner, Sir William Halcrow & Partners, London, England.

the two stations. In the sound rock of the Ceannacroc station it was considered reasonable to sit the arch on the edge of the vertical rock wall. Experience showed that it would have been preferable to allow a free bench of 2 or 3 feet in this station.

In practice the roof vault at Ceannacroc was excavated to 5 ft. below the arch springing and holes 8 ft. deep below this were drilled around the station excavation line at 6 in. centres; a further row 12 in. inside the excavation line at 12 in. centres was also drilled (of all these, only alternate holes in the inside row were charged and fired). At Glenmoriston, in view of the poor quality rock, it was decided to increase the bench under the springing to beyond that suggested after Ceannacroc and a free bench of 4 ft. was allowed for.

The decision to leave the rock walls at Ceannacroc unlined seemed to be satisfactory. It was considered that major rock movements were unlikely though some spalling was possible—in order to prevent this some parts of one gable wall had been sprayed with gunite over wire mesh. As Mr. Guthrie Brown suggested this wall was rather oppressive though the Author felt that the side walls, behind the crane columns, which were well illuminated were quite acceptable. It will be interesting to see how the rock stands up to the test of time.

Mr. Guthrie Brown's figures for station volume per MW. are interesting; it would seem difficult to draw up rules for a minimum size of underground installation. Two small installations by the North of Scotland Hydro-Electric Board which are below ground are Morar Generating Station 0.75 MW. and Mullardoch Generating Station 2.4 MW. The former is more in the nature of a cavern excavated out of a rock face—the head is only 16 ft.—the latter is at the foot of a shaft and is used to generate the power from water passed from the storage reservoir at Mullardoch to the headpond reservoir at Benevean on the Affric Scheme. These do show that in special circumstances even small installations may best be sited underground.

In reply to Dr. Colebrook's question on the possibility of revising the site of the Glenmoriston power station in view of the poor rock: excavation in the tailrace tunnel had in fact revealed better rock downstream of the station but it had not been found until too late, when the change would have meant altering the characteristics of the machines, which had already been ordered. The shaft put down was not only exploratory for it had been located on the site of the proposed intake and thus its cost had been absorbed in that of the permanent works.

The gradient of 1 in 50 at the upstream end of the Glenmoriston tailrace tunnel was originally provided in order that rail haulage might be used. The designs were prepared on this assumed gradient and it was not considered advantageous to alter the gradient when it was known that rubber tired vehicles would be used. These would have allowed a gradient up to about 1 in 9 without adopting winch assistance. No troubles had arisen due to expelling the air from the roof; the maximum design rate of load acceptance was 5 MW. per min.

In dealing with sections of poor rock in the tailrace tunnel at Glenmoriston; it had originally been intended to line such sections with concrete. Time, however, had prevented this and the rock bolting used, together with concrete placed using the Aliva Gun, appeared to be quite satisfactory when examined after 4 months operation. There appeared to be no sign of any spalling or scour and the concrete was very hard.

The Author is unable at present to put a cost on the economics mentioned at Cluanie and Loyne Dams. At the time of construction of these dams there was a serious shortage of cement and, even working with Trief concrete, some Portland cement had to be imported from Belgium and Germany. Some idea of the saving can be seen by the Contractor's offer in his tender of a reduction of £2 per ton for the substitution of Trief cement for Ordinary Portland cement. The Author's experience in the use of Trief concrete is limited to the works described and he cannot express a view as to the minimum economic size of dam for the use of Trief concrete as he has not at his disposal details of costs which were available to the Contractor. If attempts now being made to dry the slurry after grinding were successful, then this could be done at a central site and the expensive plant at the dam site would not be necessary.

A number of questions posed by Dr. Colebrook and Mr. Lawton have been dealt with in the course of the contributions by Mr. Aitken and Mr. Milne. The Author will attempt to answer the remaining points.

It was hoped that eventually these two underground stations, together with two surface stations in the adjoining Glen Garry would all be controlled from the sub-station at Fort Augustus—for this reason the accommodation for personnel had been somewhat austere. The siting of the transformers did not rest entirely with the civil engineering consultants: the economies of cables in ducts at 132 KV as opposed to 11 KV, the necessary switch gear rooms, oil tanks and fire protection measures all influenced the choice of site for the transformers.

The Author regrets that he cannot give Mr. Lawton figures to substantiate the savings obtained by using facing blocks. He is quite certain that their use was well justified due to the speeding up of the work when joiners were scarce. Masons were required for setting the blocks but one mason could supervise a number of labourers at this task. The attractiveness of the blocks has been commented on frequently and they are certainly built to last without losing their appearance.

Mr. Milne's remarks on page 142 cover quite fully the remedial measures taken to support the rock walls in Glenmoriston Power Station and the Author trusts that no further explanation is necessary.

The cost of salmon measures in Glenmoriston is difficult to assess but a rough figure of £150,000 ($420,000) might not be far out. This is solely the cost of screens and structures, fish lock, special sluice gates etc., fish heck, trap and holding ponds and river works to enable the fish to swim where the flow has been reduced.

There are many other factors which affect the cost of the scheme such as:

Increased compensation water.
Special plant arrangements (e.g. provision of a 4 MW machine at Ceannacroc).
Special precautions during construction to prevent river pollution, temporary drying out of the river for river works etc.
Fish Hatchery and cost of staff and accommodation of Fishery officers.

In comparison with the cost of the scheme (£10,000,000 or $28,000,000) this was quite a modest sum but it must be borne in mind that the River Moriston is not considered to be one of the major salmon rivers of Scotland and the provisions in the neighbouring Glen Garry were much more considerable.

Forced ventilation had not been installed in either station though space had been made available for the necessary plant should it be desirable. At Ceannacroc measurements of humidity and temperature were being recorded inside and outside the station and at various points within the station.

The remedial treatment required at Ceannacroc was negligible compared with that at Glenmoriston. No comparative unit costs are available at present. The arch roofs of the power stations had not been cracked by the blasting of the dumpling though some pitting had occurred: this was hardly discernable from the turbine floor.

PERFORMANCE OF COGSWELL AND SAN GABRIEL ROCKFILL DAMS[a]

Closure by Paul Baumann

PAUL BAUMANN,[1] F. ASCE.—The writer would be very happy indeed if he had a direct and unequivocal answer to all the pertinent and constructive questions raised by the discussors. Unfortunately, however, questions raised in regard to the actual physical performance of the rock in the interior of the dam must of necessity be answered indirectly, that is to say, in light of circumstantial evidence. One feature, however, should be made clear to avoid further misunderstanding, namely that Cogswell Dam itself is not considered to be of critical height but that the lesson learned through its performance should be duly considered by engineers engaged in the planning and design of rockfill dams of materially greater height. This happens to be the trend at the present time. There is a great challenge in the harnessing of formidable energies through the construction of major dams but the responsibility is, needless to say, even greater. The writer's paper, so ably and gratifyingly discussed, was prepared in the spirit of this responsibility. It does sound a warning in regard to the possibility of transformation of a loose rockfill into a dense rockfill with increasing height. In the writer's opinion, this possibility had not been duly considered heretofore.

Before referring to each of the discussions separately a general statement concerning pertinent factors in the construction of Cogswell Dam may be in order. It is hoped that some of the questions, implied rather than expressed by discussors, may be answered at least in part.

The writer was aware of and intrigued by the performance of Cogswell (then San Gabriel No. 2) Dam during its construction before his employment in October 1934 with the Los Angeles County Flood Control District. He strongly felt that the causes of measured shrinkage during construction and additional large settlement when the completed embankment was wetted by some 15 inches of rainfall, merited thorough investigation, study and analysis. Thus, immediately after his employment with the District in charge of dams and water conservation activities, the writer assigned his principal assistant, the late N. B. Hodgkinson, A. M. ASCE, formerly in charge of the District's design division, to undertake such investigation and report thereon. After examination of several hundred inspectors' reports, covering all phases of construction activities from the quarrying of the rock to its final placing in the dam, it was concluded that 1) the size and grading of the rock all the way down to quarry dust closely conformed to specifications; the average volume per truck load of Class A rock placed throughout the main fill of the dam was 11.5 cu. yds. and the corresponding weight 33,600 lbs.; the average weight per truck load of

a. Proc. Paper 1687, June, 1958, by Paul Baumann.

1. Asst. Chief Engr., Los Angeles County Flood Control Dist., Los Angeles, Calif.

quarry dust was estimated at 1000 lbs. leaving a net weight of 32,600 lbs exclusive of quarry dust. These are the figures used by the writer in arriving at densities after consolidation. Frankly, the porosity of 40% exclusive of quarry dust which follows from the above figures and the specific gravity of 2.80 as an average of 211 tests, appeared rather low at first glance. However, even if the porosity had been in error by possibly 10% and therefore had been 44% instead of 40%, the conclusion in regard to impermeability at the base of the dam would remain substantially unchanged. The dry density of the crushed material (which of course includes rock dust) and quarry dust would be reduced to 123 p.c.f. or slightly more than the 120 p.c.f. required in the center core section of San Gabriel Dam (No. 1). The rate of percolation for a unit gradient corresponding to this density was found to be 3.0 ft. per year. Hence, the question of whether or not a rockfill dam will become impervious from the base upward with increasing height does not hinge so much on initial densities as it does on the size and uniformity of the rock. Mr. O'Shaughnessy, as quoted by Mr. Spielman, has put the wisdom of the fundamental requirement for dry construction of rockfill dams in a nut shell, namely that the rock should be uniformly large. While crushing of points in contact would still result in shrinkage, it would be small as compared to that of rock varying in size such as specified for Class A at Cogswell Dam. The reason is simple. In addition to point crushing, a mixture of large and small rocks gives rise to what might be called the "nut cracker effect". Small rocks wedged between large rocks can readily be crushed. It very seldom works the other way. Therefore, to minimize such crushing and production of fines associated therewith, the small rocks together with initial fines should be sluiced into the interstices at time of placing. In other words, sluicing of such mixtures is indispensible if excessive production of fines after placing is to be avoided.

Unfortunately, the ideal rockfill dam, consisting of uniformly large rock, is seldom economical because of the smaller material unavoidably produced in blasting that will have to be separated and wasted. The writer knows of no major rockfill dam built exclusively of uniformly large rocks. The cost of sluicing a mixture of rock with a minimum of waste evidently has so far been considered economically advantageous. Nevertheless, the possibility of creating a zone of low permeability in the interior even of a sluiced dam in excess of say 350 feet in height should always be carefully investigated. "Low permeability" should be interpreted as <u>not</u> free draining. Hence, even if some channels here and there should remain open the respective percolation could be far more critical than that through a free draining mass.

Mr. Brown deplores the absence of physical demonstration of impermeability within the body of a high rockfill dam and so does the writer. If San Gabriel Dam (No. 1) had been built in a similar manner as Cogswell Dam, chances are that such demonstration would have been convincingly furnished, even if the rock there had been of equal quality as that at Cogswell. Actually not all of the rockfill dams built in the past have been favored with the same high quality of rock as that at Cogswell Dam, nor will future dams necessarily be so favored. The original estimated leakage of 130 c.f.s. through Cogswell Dam under full head has been revised to 105 c.f.s. Based on free draining conditions and the timber facing the controlling "water stop", 200 c.f.s. were anticipated. There were no means of locating the free water surface for turbulent flow or the phreatic surface for laminar flow. The void space between the extra large (Class B) rock at the downstream face, especially in its lower part, prevented any observation of the discharge face of flow or

percolation through the interior zone of the dam. Furthermore, the tightest part of this zone would, in view of normal pressure distribution in vertical sections, be below streambed. However, a few corresponding water surface elevation and leakage points taken from the official record, may be of assistance in arriving at some conclusion as to the effect on leakage of consolidation of the rock mixture.

Water Surface Elevation	Date	Leakage (c.f.s.)	
2177	12-10-37	1.00	
2236	2-1-38	6.00	
2337	2-26-38	20.00)	Over 20" of rainfall on exposed part of timber facing
2350	3-1-38	40.00)	
2397	3-2-38	105.00)	
2312	3-10-38	6.50)	Timber facing "sealed" by sediments
2357	3-20-38	13.00)	
2376	6-30-38	15.00)	
2170	8-16-38	0.50)	

For a free draining rockfill, the leakage through the dam should be in direct proportion to the pressure on the facing the area of which can be approximated by a trapezoid 410 feet wide at the top, 185 feet at the bottom and 260 feet in height. The leakage Q' per unit width at the maximum height of the dam

$$Q' = c\,\frac{62.5H^2}{2}$$

in which c = coefficient of leakage (affected by the swelling of the timber facing)

Since the pressure distribution is triangular it follows that 1/2 Q' must pass through the bottom 1/3 of the section. Taking the trapezoidal shape of the facing into account it follows that the total leakage for maximum head

$$Q_{max} = 105 = 7800\ cH^2 \text{ and}$$

$$c = 0.019 \times 10^{-6} \text{ ft/sec}$$

From the above tabulation it may be noted that for the water surface at elevation 2337, that is at 3/4 H the leakage was 20 c.f.s. whereas under this head the coefficient

$$c = \frac{20}{4600H^2} = 0.00614 \times 10^{-6} \text{ ft/sec}$$

Thus it follows that about 75% of the maximum leakage passed through the top 1/4 of the facing.

If, conversely, percolation through the consolidated rockfill had been controlling the leakage and the rockfill had become a quasi homogenious fragmental mass, similar to Zone 3 at San Gabriel Dam then, as shown in the flow net[1] 1/2 of Q" would have passed through the top 1/3 of the maximum section of unit width. Approximating the flow through the top, middle and bottom 1/3 of the section by 1/2, 1/3 and 1/6 of Q" respectively then

$$\frac{q''_t + q''_b}{2} \times H = Q''$$

in which q''_t = rate of flow at top

and q''_b = rate of flow at bottom

Applying the trapezoidal shape of the face as before the rate of flow at the top, middle and bottom

$$q_t = 717.5\ Q''/H$$

$$q_m = 297.5\ Q''/H$$

and $q_b = 46.3\ Q''/H$ (due to averaging flow through bottom 1/3)

There follows

$$Q_{max} = 313\ Q'' \text{ and for } (2)$$

$$Q'' = \frac{5}{16} KH \quad ;$$

$$Q_{max} = 105 = 98\ KH$$

Therefore, for H = 260'

$$K = 0.004125 \text{ ft/sec} = 130{,}000 \text{ ft/yr}$$

For the water surface at 3/4 H and Q = 20 c.f.s.

$$Q = 20 = 157.5\ Q'' = 48.2\ KH$$

and $K = 0.001595$ ft/sec = 38,700 ft/yr

Hence, in this case the permeability for the bottom 3/4 of the dam would be about one-fourth of that for the entire height of the section.

While no definite conclusions could be drawn from the above one thing appears certain, namely that the lower part of the dam was less pervious than the upper part.

Mr. Brown's comparison of the massive core section at San Gabriel Dam and a hypothetical one 10 to 20 feet thick with correspondingly steep faces merits close attention by the profession. The inner flexibility and unpredictable attendant movements inherent with all rockfill dams call for core dimensions commensurate with differential (unbalanced) pressures which must be taken up in shear by the saturated core material. Again, this problem grows rapidly with the height of the dam. The effect of weighting on the roughly 1 to 1 slopes of the core section of San Gabriel Dam (No. 1) was treated as prestress by the writer and its significance was analyzed.(3)

Mr. Spielman ponders the qualification of San Gabriel Dam as a rockfill dam. Although he took an active part in the revision of the original design he apparently forgot that the "large central portion" is compacted quarry rock with pieces up to 9 in. in size rather than compacted earthfill. The earthfill part in Zone II amounts to only about 10% of the total volume. Hence, "rockfill dam" seems to be the proper classification. Incidentally, Mr. Spielman also deserves considerable credit for the ingenious design of the temporary timber facing that performed to full satisfaction at Cogswell Dam.

The writer agrees with Mr. Spielman and so stated, namely that the laminated facing and flexible connection to the cut-off wall by means of the floater slab might well have performed satisfactorily under normal settlement conditions. However, the fulcrum effect along the abutments could still have been critical even if settlements had been considerably smaller. To overcome this effect would have required much wider benching between the cut-off wall and the cliff face so as to rest the entire floater slab on a tapering section of "small round stone" with its thickness roughly proportional to the distance from the hinge. (See Fig. 4d) This would have required greatly increased rock excavation. Mr. Spielman's conclusion that "there is no justification for more than a simple, heavy reinforced-concrete slab" is therefore strongly concurred in.

Mr. Lawton's question regarding arching of the rock in the V-shaped gap adjacent to the right abutment must again be answered based on circumstantial evidence as follows: (1) The higher lift naturally caused greater segregation of the rock, the larger pieces running ahead of the smaller ones. At the bottom of the gap the rocks hit a pocket which actually was semi cone-shaped, in that only the upstream portion of the dam was affected by the delay in the cut-off wall construction while progress on the portion downstream therefrom continued as before. Thus, with large rock falling into this semi conic pocket arching could hardly have been avoided. (2) The maximum settlement due to rain and subsequent sluicing occurred in this V-gap area and is in plain evidence today. In fact Fig. 12 shows it as a sag in the cat walk at the top of the paneled facing slab.

Mr. Lawton's statement that "ample sluicing water is imperative" cannot be too strongly emphasized, especially where rock mixtures are concerned such as described under Class A rock. The main part of rockfills are predominantly of this kind of mixture.

As to the wisdom of lifts up to 50 to 75 feet instead of the 25 feet used at both Cogswell and San Gabriel Dams the writer can see no decisive advantage in higher lifts, whether placed dry or wet, but can readily conceive of some disadvantages. These are due to (1) segregation of larger from smaller rocks previously mentioned; (2) lesser benefit from equipment compaction and (3) increased break-down and production of fines due to greater impact force.

The elastic behaviour of San Gabriel Dam (No. 1) under full water load and subsequent emptying of the reservoir can only be attributed to the monolythic properties acquired by the massive, center core section of screened and compacted quarry rock. In addition to the initially created high density this section's roughly 1 to 1 slopes are weighted and laterally supported by loose rockfill and therefore are prestressed as outlined above. Under these conditions deformations can become elastic. This further strengthens Mr. Brown's statement regarding thin cores in rockfill dams. In brief a massive core may well be the strongest and a thin core the weakest part of a rockfill dam.

As to comparative cost of maintenance, repairs and replacement at the two dams the records show that during the last ten years some $25,000.00 average were spent at each dam each year. Since the initial costs were, in round figures, $4.5 and 17 million for Cogswell and San Gabriel Dam, respectively, the corresponding yearly cost of maintenance, repairs and replacements amounted to 0.56% and 0.15% of the initial investment.

Messrs. Fetzer and Knight enumerate five "assumptions" upon which the computation of the density of 132 p.c.f. was based. In light of the introductory

statements in this closure the first three items quoted can not be classed as assumptions, whereas the fifth item is in that category only so far as possible axial arching at the base is concerned. In the absence of such arching the pressure at the base of a column of rock is twice the average and so would normally consolidation be. In a symmetrical monolyth of triangular cross section the major and minor principal stresses follow circular arcs and radial lines, respectively, around and through the apex of the triangle so long as the loading is due to gravity only. Normal stresses are uniform along horizontal planes. Since shear is zero along principal stresses there is no shear along the axis of symmetry. In comparison the pressure distribution along horizontal planes of a symmetrical triangular embankment dam of compacted earth is directly proportional to the height of the column above it. Hence, basically it is triangular. However, due to plastic deformations the stress distribution has never been uniquely established but has merely been approximated by applying a "compaction factor" which is the ratio of horizontal and vertical stress. Therefore all stresses at points on the outer slopes are zero and the maximum concentration of stress or pressure occurs in a zone at and adjacent to the axis of symmetry. In general this no doubt applies to rockfill dams as well. The writer referred to this zone as the interior portion of the dam.

The writer agrees that there is room for debate on item four which is an assumption entirely based on judgment and not on observation. In the well graded mixture of Class A rock, 40% varied in size from quarry chips to 1000 pound rocks. Thus the 25% of the total assumed to have been crushed was generally smaller than 1000 pound rocks, more of the order of 500 to 600 pounds maximum, that is rock of about three cubic feet in volume. Considering the fact that 60% of the total volume was rock between 1000 and 14,000 pounds the "nut cracking" of 25% of generally much smaller rock does not seem to be an unreasonable expectation.

The test fills at San Gabriel Dam (No. 1) should not be used as a criterion of density at the base of Cogswell Dam. The dry density of the fines of 89 p.c.f. quoted by Messrs. Fetzer and Knight were found in the test fill, 25 feet in total height, between its surface and twenty feet below the surface. Hence, the self-compacting effect under some 280 feet of fill was not reflected in this test fill. The shrinkage in the dam for the first 25 feet lift was 1.75%. Thus the composite dry density had increased from 105 to 106.8 p.c.f. which would have no effect on permeability.

The uncertainty of stress distribution in an embankment dam of compacted earthfill is much greater for rockfill. There is simply no way of even approaching the exact distribution of internal stresses in a rockfill dam. As stated by Messrs. Fetzer and Knight the weight of the fill as well as the applied load is, to a considerable extent, probably carried to the foundation by point to point contact. However, this does not mean that only the larger rocks are in contact unless the fill consists of nothing else. In a mixed fill some of the smaller pieces will necessarily get between the larger rocks, especially if placed dry. Hence, there will be rock crushing first, point crushing second and compression of filler materials last by general order of sequence as visualized by the writer. The compressed filler material, whether sluiced or crushed into the interstices will lend stability to the larger rocks subjected to excentric contact loading. It would be interesting indeed if means could be devised for direct observation of the goings-on in the interior of a rockfill during construction and subsequent application of water load. It would eliminate the need of much speculation.

In elaborating on some of the construction features Mr. Porter has furnished much valuable supplemental information. Of special interest is the description of the completion of the cutoff wall on the right abutment. In the paragraph next to the last Mr. Porter enumerates what might be called the cardinal principles of rockfill dam construction with special emphasis on the importance of sluicing. The writer is in full agreement with these principles but would like to reiterate his word of caution in regard to the ever increasing height of rockfill dams, even if thoroughly sluiced, namely, 1) the face slopes should be decidedly flatter than the angle of repose of the rock; 2) if a water tight concrete facing is intended a temporary timber facing pending cessation of settlement may be advantageous; 3) if a watertight core is intended it should add to the stability of the dam rather than relying on the dam for its support; and 4) assurance of free drainage of the fill should not be taken for granted but should be carefully analyzed.

REFERENCES

1. Design and Construction of San Gabriel Dam No. 1 by Paul Baumann, M. ASCE, 1942 Transactions ASCE Vol. 107, p. 1631.
2. Ibid., p. 1630, Eq. (11b).
3. The Significance of Prestress in Embankment Dams Proceedings, 2nd Conference on Soil Mechanics and Foundation Engineering, Vol. VII, p. 92.

ROCKFILL DAMS: THE BERSIMIS SLOPING CORE DAMS[a]

Closure by F. W. Patterson and D. H. MacDonald

F. W. PATTERSON,[1] and D. H. MacDONALD,[2] M. ASCE.—The authors wish to thank Mr. Lawton for his discussion and in reply, to offer the following comments:

The layer of varved clay and silt which was found between two thick layers of glacial till in the north bank deposits is appreciably coarser in grain size than the varved clays found at Steep Rock Lake and described by R. F. Legget.* Whereas the percentages of clay size particles in the Steep Rock deposit range from about 25 to 85, the similar values for the Bersimis deposit range from 0 to 20 per cent. Moreover, the entire north bank deposit of till and varved clay existed in an overconsolidated state with a low sensitivity, in contrast to the normally consolidated state and the highly sensitive structure of the Steep Rock clays. Because of this absence of high sensitivity, none of the unstable conditions which were found at Steep Rock Lake, were observed or would be expected to occur upon remoulding of the Bersimis varved clay.

In the construction of the main rockfill on the north bank care was exercised to ensure that the transition layers separating the rockfill from the foundation deposits were not damaged. The rockfill immediately overlying these transitions was built with the front of the stage progressing towards the north bank. Where this face met the steep portion of the abutment bordering the river it was necessary to support the transition layers on the steep slope with a berm of rockfill placed and compacted in thin lifts and sluiced only in part. This berm provided stability for the transition layers and provided access for their construction. Aside from this small portion the entire rockfill was sluiced in the standard manner. The sluicing water percolated down through the rockfill and on the north bank it flowed directly into the river or it was collected in a ditch along the downstream toe which was lined with layers of transition material. Although the water flowed over and through the transition layers the sluicing jets were not played directly on them. This operation did not result in any known detrimental effects.

---

a. Proc. Paper 1740, August, 1958, by F. W. Patterson and D. H. MacDonald.

1. Sr. Engr., Hydr. Div., H. G. Acres & Co. Ltd., Niagara Falls, Canada.
2. Geotechnical Engr., Technical Div., H. G. Acres & Co. Ltd., Niagara Falls, Canada.

* Legget, R. F. (1958), "Soil Engineering at Steep Rock Iron Mines, Ontario, Canada", Proceedings of Institution of Civil Engineers, Paper No. 6304, Vol. 11, pp. 169-188.

The use of calcium chloride in the construction of the impervious core did not, to the best of the authors' knowledge, adversely affect the quality of the core in any way. The densities obtained during compaction were as high when calcium chloride was used as they were without it. In view of the fact that the till used for the Bersimis core contained no more than about 5 per cent of clay size particles it is reasonable to conclude that the calcium chloride would not change the structure of the compacted soil. However, in the case of impervious materials containing much higher quantities of clay size particles, and particularly in cases where these particles may be highly active clay minerals, the use of calcium chloride followed by a leaching process during a steady seepage state might result in the creation of a sensitive soil structure within the core. Although it is possible that even in such cases the use of calcium chloride may not prove to be detrimental, the authors would not like to recommend this practice in these cases without studying the possible effects by laboratory testing.

ROCKFILL DAMS: THE PARADELA CONCRETE FACE DAM[a]

Discussion by Armando da Palma Carlos

ARMANDO da PALMA CARLOS.[1]—The authors have presented a comprehensive and valuable paper on Paradela Dam, the highest rockfill dam in Portugal. This discussion presents data on the second highest rockfill dam in Portugal, the 207 ft. high Salazar Dam. Salazar Dam, though much smaller than Paradela Dam, is of special interest because of the use of a steel facing as the watertight membrane.

Salazar Dam forms part of the Vale do Sado Lower Course Irrigation Scheme, which also includes the production of hydroelectric power. Full operation began in 1949. The project includes two large storage reservoirs; Pego do Altar Reservoir (Salazar Dam) with a storage capacity of 76,200 acre feet;(1) and Vale de Vaio Reservoir (Trigo de Morais Dam) with a storage capacity of 51,100 acre feet. The Trigo de Morais Dam is a 167 ft. high earth and rockfill dam, watertightness being obtained by means of a central bituminous concrete membrane.

Main Features of Design

Salazar dam is a rockfill structure provided with a steel facing, laid on a thin concrete layer which covers a layer of masonry; it possesses a cutoff wall with an inspection gallery, from the interior of which the grouting operations of the foundation were carried out. Its main characteristics are:

Maximum height above the cutoff wall foundation	207 ft.
Rockfill structural height	184 ft.
Length of crest	630 ft.
Crest width	16.4 ft.
Slopes:	
Upstream face	1.25 on 1
Downstream face (3 berms—6.6, 8.2 and 9.8 ft. wide)	1.4 on 1 between berms

Geological Features of the Dam Site

The devonian, essentially silicious and argillaceous shales which compose the rock at the dam site are intensively folded and fractured, comprising quartz inclusions in the form of veins and nodules. The low strength of the foundation called for a dam with a wide base.

---

a. Proc. Paper 1747, August, 1958, by Luis Henrique Gomes Fernandes, Edgard de Oliveira and Nuno de Vasconcelos Porto.

1. Head of the Reclamation Div. of the Hydr. Dept., Portuguese Ministry of Public Works.

A large vertical fault, about 13 feet thick, occurs in the river bed, its direction being shown by Fig. 1; this fault contains an argillaceous mass with large and small blocks of the early shale. Such geological conditions led to the adoption of the following construction methods for the purpose of avoiding piping:

At the fault zone, the cutoff wall had to be carried to a depth of (-26.00), which corresponds to half of the hydrostatic head.

Downstream from the cutoff wall, a filter was constructed formed by the following 5 layers, each 1.3 feet thick: fine sand (bottom layer), coarse sand, stonecrusher waste, fine crushed stone, coarse crushed stone (top layer). Thus the voids increased from the bottom to the top layers.

Cutoff Wall, Drainage Gallery and Pumping Well

The cutoff wall is located under the upstream part of the embankment, between the steel facing and the foundation, and contains an inspection gallery. The drainage gallery, embedded in a drain formed by small stones, was constructed in the deepest zone of the foundation, ending in a downstream pumping well. The access to the cutoff wall gallery is made possible by two inspection galleries driven through the valley slopes, Fig. 1.

Rockfill

The cross-section of the dam is shown on Fig. 2. The embankment consists of 485,000 cu. yds. of porphyry rock. The density of the rock was 2.665 and the percentage of voids in the rockfill was 34.4. The largest blocks used in the rockfill weighed 8 to 9 tons.

The rockfill was dumped and partly hand-placed in the central embankment in layers of approximately 9.2 ft. in depth. Cranes were used for placing the large blocks in the upstream embankment, in the downstream toe and in upstream face, as well as for constructing the upstream masonry. This cement mortar masonry is of variable thickness, ranging from 6.6 feet at the bottom to 2.6 feet at the top, its volume being 16,475 cu. yds. The surfacing was made with a concrete layer with an average thickness of 8 in., on which the steel facing sheets were laid. In both upstream and downstream faces corrections had to be introduced to take into consideration the settlements[2] which certainly would occur, not only during the construction but also thereafter, mainly during the filling of the reservoir.

The following corrections were allowed to compensate for estimated settlements occurring after the construction: maximum crown of 2 ft, crest convex in plan, with a deflection of 1.64 ft., and upstream face concave in cross section.

Steel Facing

This type of membrane has been chosen in view of its strength, flexibility and watertightness. Its main features are presented in Fig. 3 and described below:

Steel sheet facing, clamped along the cutoff wall, free on its upper part.

Dimensions of the steel sheets: 24.6 x 8.2 ft; 5/16" thick up to one third of the dam height, 1/4" from there upwards.

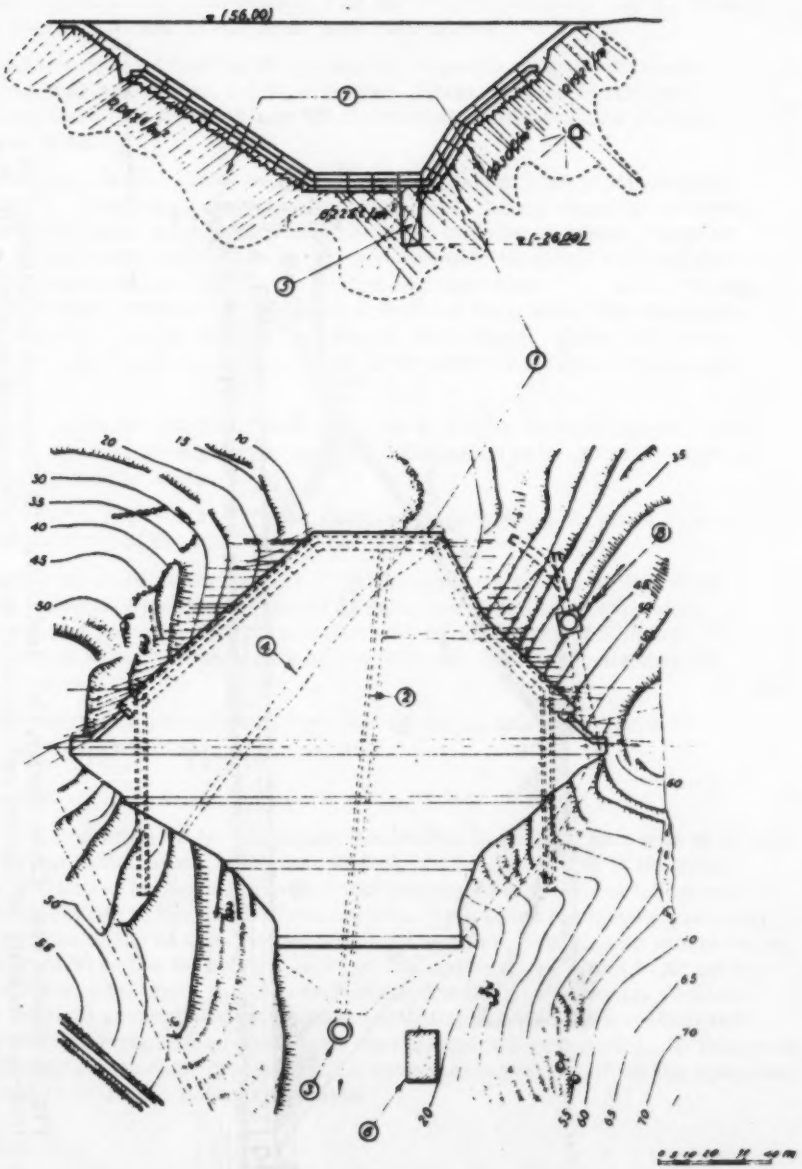


Fig. 1. Salazar dam. Front view and plan. (1) Cutoff wall. (2) Drainage gallery. (3) Pumping well. (4) Geological fault with a filter at the downstream side of the cutoff wall. (5) Deep zone of the cutoff wall at the intersection with the fault. (6) Power plant. (7) Grout curtain. (8) Intake tower.

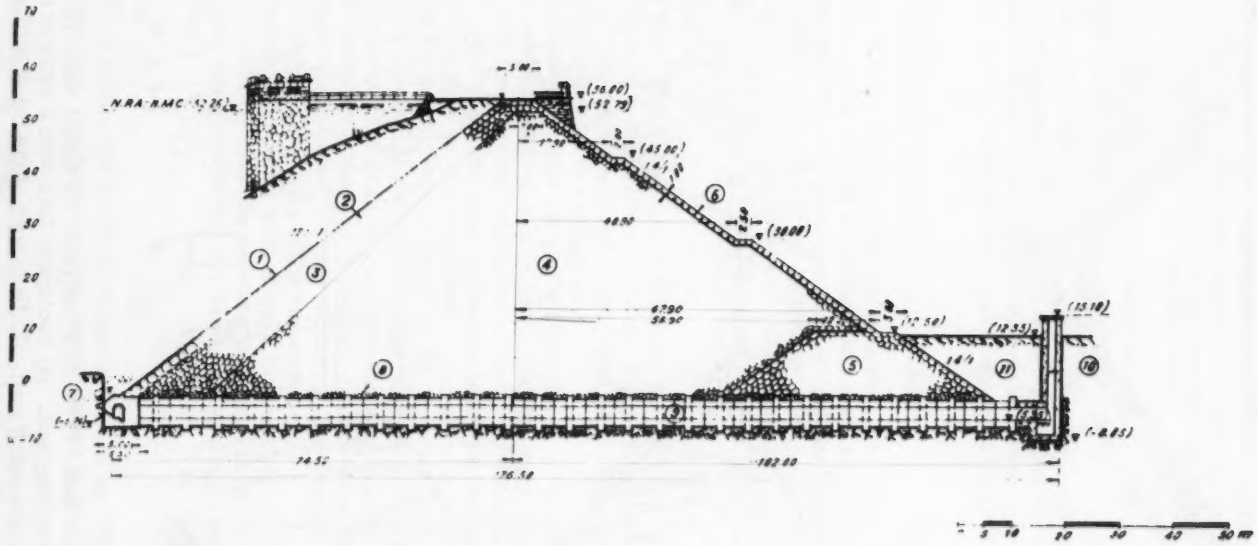


Fig. 2. Salazar dam. Cross section. (1) Steel facing. (2) Cement mortar masonry. (3) Upstream rockfill. (4) Central embankment of dam. (5) Downstream rockfill toe. (6) Dry laid masonry. (7) Cutoff wall. (8) Drainage gallery drain. (9) Drainage gallery. (10) Pumping well. (11) Tipped fill.

Sheets horizontally butt-welded, with 45° chamfered lower edges. Welding seams, at least 3 mm thick, upon butt-plates, 1" x 1/8".

Expansion joints following the maximum slope direction, with semi-cylindrical butt-plates, 1.3 ft. diameter. Sheets welded to the butt-plates, the edges of which are 60° chamfered; welding seams at least 3 mm thick.

Connections to the cutoff wall by means of 300 mm UNP steel sections, with the lower edges stopping at a distance of 10 mm, from the concrete, this space being filled with plastic bitumen in order to insure complete watertightness; the triangle comprised between the cutoff wall and the membrane was also filled with molten asphaltic concrete. Use of strong gussets plates where the U-shaped structural steel elements change direction, in order to prevent ruptures at such points. Along the beams, anchor bolts 6.6 ft. in length and 1" in diameter, 1 ft. apart from each other.

Anchor bolts for the steel sheets, 4.1 ft. in length, 1" in diameter. Provision allowing an expansion up to 3". Expansion nuts and rings covered by protection helmets.

Use of 2" x 5/16" bars between each two anchor bolts, in order to prevent buckling of the steel sheets between those bolts.

Behind the steel facing, drains 2" in diameter connected to the cutoff wall inspection gallery, each one of them connected to an expansion joint. The concrete face was coated with priming coat of Flintkote No. 1 and two waterproofing coats of Flintkote No. 5 before installing the steel face.

Anti-corrosive protection of the steel facing by means of Flintkote bituminous emulsion.

Settlement

During construction the maximum settlement in Salazar dam was 29.9" and it occurred in the downstream face at 60% of the total height of the axial cross section. The more important settlements after the steel facing was installed occurred in the same cross section, their value increasing gradually from bottom to top of dam. After ten years service, settlements measured at crest level were the following: 14.9" on the upstream side and 17.4" on the downstream side. Both values were obtained with full reservoir. Besides these vertical settlements, some horizontal displacements were observed under the same conditions (ten years service and full reservoir). At the crest, the following horizontal displacements were measured: 14.0" on the upstream side and 13.6" on the downstream side.

Leakage

Leakage, either through the steel facing, on account of three fissures having appeared in the welding of the semi-cylindrical butt-plates of the joints, or at the connection of the facing to the cutoff wall, was only observed during the first and the second fillings of the reservoir. Maximum observed leakage was 145.6 U.S. gallons per minute, most of it (79.2 U.S. gallons per minute) resulting from the above mentioned fissures.

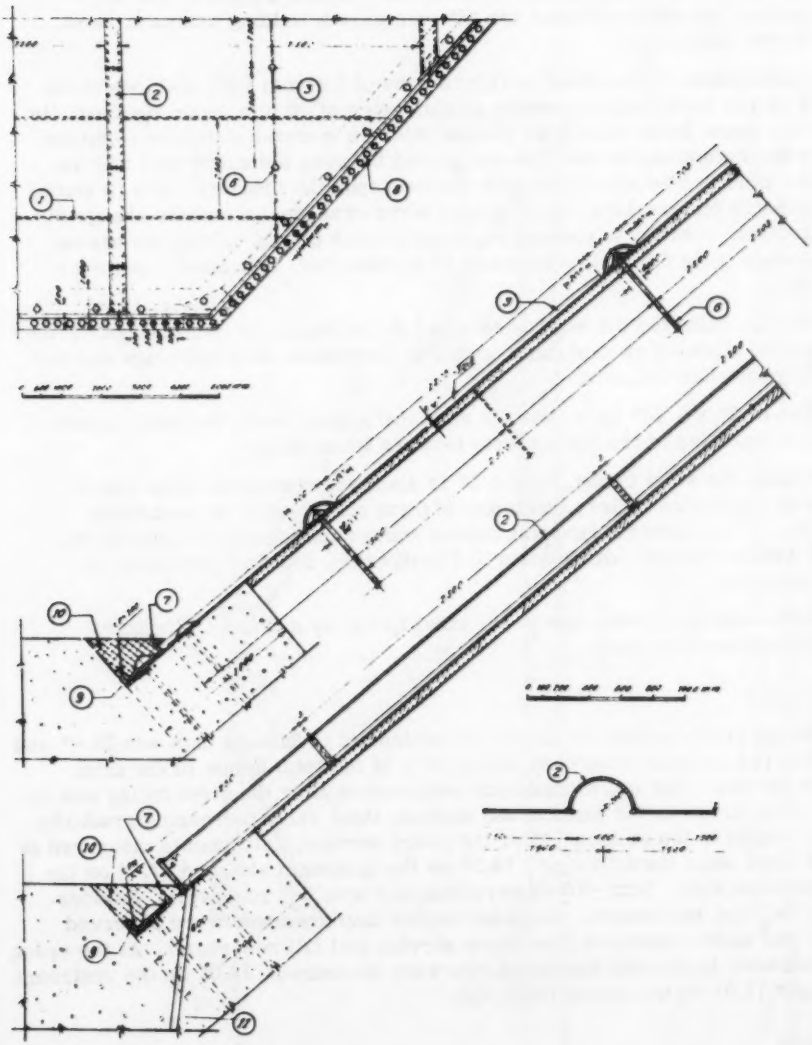
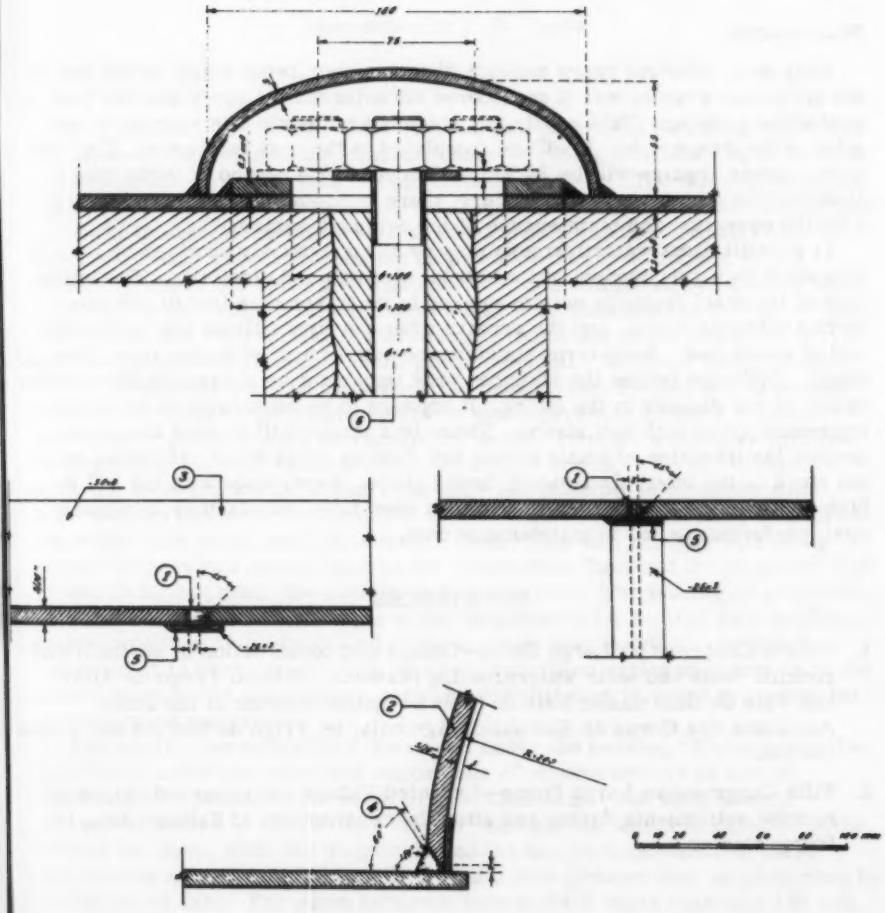


Fig. 3. Salazar dam. Steel facing. (1) Horizontal connections between the sheets. (2) Butt-plates of the expansion joints. (3) Reinforcing bars. (4) Welding of the steel sheets to the expansion joint butt-plates. (5) Lower butt straps in the horizontal connections between the plates.



(6) Anchor bolts of the steel sheets. (7) UNP steel sections connecting the steel plates to the cutoff wall. (8) Anchor pieces in the cutoff wall. (9) Bituminous joint at the connection to the cutoff wall. (10) Asphaltic concrete. (11) Drains connecting with the cutoff wall.

The existence of drains, in communication with the cutoff inspection gallery, made it possible to locate immediately the fissures. These were nearly completely sealed with tarp cloth slowly lowered from the crest along the joints. Leakage was in this way greatly reduced even before final repairs by welding. After these repairs, which were done before the third filling of the reservoir, no other leakage was observed through the steel facing, from there on perfectly watertight.

Maintenance

Only now, after ten years service, the reservoir being empty at the end of the irrigation season, was it considered advantageous to apply another coat of protective painting. This repair has already been made this year up to one third of the dam height. It will be completed in the next two years. The total cost of these repairs will be $4.500, extended over a period of three years. Besides this maintenance expenditure, there is another small annual charge with the operation and maintenance of the drainage system.

In rockfill dams watertightness may be advantageously obtained by the use of a steel facing on account of its special strength and elasticity. The initial cost of the steel facing is not considered to be higher than that of the reinforced concrete facing, and the repairs after the first fillings are convenient and of small cost. Long term maintenance cost of a steel facing is moderately small. Drainage behind the facing is very important as it permits the locating easily of any damage in the facing. It appears to be advantageous to protect expansion joints with butt straps. These butt straps will protect the joints against the intrusion of small stones and floating twigs which otherwise might get stuck in the intervals between facing slabs. Experience with the 207 ft. high steel faced Salazar rockfill dam has been most satisfactory in capital cost, performance and in maintenance cost.

REFERENCES

1. Fourth Congress on Large Dams—Design and construction of earthfill and rockfill dams and their waterproofing blankets. Salazar (Pego do Altar) and Vale de Gaio dams, Vale do Sado Irrigation Scheme of the Junta Autónoma das Obras de Hidráulica Agrícola, by Trigo de Morais and Palma Carlos.
2. Fifth Congress on Large Dams—Expected Values and observed values in rockfill settlements during and after the construction of Salazar dam, by Palma Carlos.

ROCKFILL DAMS: DESIGN OF COUGAR CENTRAL CORE DAM[a]

Discussion by J. B. Cooke

J. B. COOKE,[1] F. ASCE.—Mr. Thurber has presented a concise and valuable paper on the engineering considerations of geology, foundation, materials and design for a rockfill dam of unprecedented height. The thoroughness and caution in the considerations is particularly appropriate since the unprecedented height is associated with less than ideal geology and materials. The use of experience data and measurements on previous dams, mentioned by the author, stresses the value and importance of this Symposium in making such information available to all engineers engaged on planning and design of dams. The experience information is increasingly important in the selection of type and design of rockfill dams of still greater height. The availability of considerable experience data made possible through this Symposium in no way reduces the need of the pooled judgment and experience of many engineers on any specific project, such as recognized in Mr. Thurber's acknowledgments.

Regarding streambed excavation, the author states, ". . . not planned to excavate to bedrock except under the core and possibly in an exploration trench at the downstream toe of the dam". Where it is reasonably feasible to place the core on bedrock, it is done. Where it is not, designs with an upstream impervious apron, such as for Cheakamus Dam and the proposed High Aswan Dam, can adapt the rockfill dam to the site. The leaving of substantial amounts of streambed materials in the foundation of a rockfill dam is often of major economic importance and in most cases may result in a better dam. However, it is considered that a properly backfilled exploration trench at the downstream toe is a necessary part of a rockfill dam intended to be founded on streambed materials.

The careful considerations discussed under the heading "Embankment Design" emphasize the value and importance of measurements of lateral movements of points on the crest of a rockfill dam. The term lateral movement refers to the "across canyon" component, which is parallel to the axis of the dam. Both the magnitude and the design importance of these movements are considered to increase at a rate greater than in proportion to the height of dam. For some concrete face rockfill dams less than 150 feet in height, the lateral movements have been observed to be negligible. For a well designed and constructed core rockfill dam of less than about 325 feet, these lateral movements are perhaps not significant. However, to be assured that the performance of a rockfill dam is satisfactory, and to provide a basis for improved design of higher dams, it is considered that lateral measurements of points on the crest should be included as a design and operating feature of all rockfill dams of significant height.

a. Proc. Paper 1749, August, 1958, by Paul Thurber.

1. Superv. Civ. Engr., Pacific Gas and Electric Co., San Francisco, Calif.

The central core section shown on Fig. 3 of the paper indicates zones of compacted rock both up and downstream from the core. The reasons for the compacted rock sections are not expressly given in the paper. It appears that the compacted rock sections permit the utilization of the smaller quarried rock in the dam and provide some construction advantages in coordinating the core and rockfill construction. The minimum thickness of the downstream zone of compacted rock is probably a basic design requirement as a compacted transition zone. It would be of interest to know the extent to which the compacted rock zones were considered necessary for structural design reasons for this high dam. Also, the results of the test compacted rockfill would be of value. Neglecting the fact that the compacted zone permits the use of rock that might not be suitable for dumped rockfill, how do the bid prices of compacted and dumped rock compare.

At the time of writing the paper, design of the core and rockfill section was not completed. Considerable design progress has been made since, resulting in the award of the construction contract in June 1959. It would be of particular value and interest that the author bring the design history of Cougar Dam up to date.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

VOLUME 84 (1958)

AUGUST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733 (PO4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742 (PO4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758 (ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767 (WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775(IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784(IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

OCTOBER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 1798(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806(HY5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815(ST6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824(SM4), 1825(SM4), 1826(SM4), 1827(ST6)[c], 1828(SM4)[c], 1829(HW3)[c], 1830(PO5)[c], 1831(EM4)[c], 1832(HY5)[c].

NOVEMBER: 1833(HY6), 1834(HY6), 1835(SA6), 1836(ST7), 1837(ST7), 1838(ST7), 1839(ST7), 1840(ST7), 1841(ST7), 1842(SU3), 1843(SU3), 1844(SU3), 1845(SU3), 1846(SU3), 1847(SA6), 1848(SA6), 1849(SA6), 1850 (SA6), 1851(SA6), 1852(SA6), 1853(SA6), 1854(ST7), 1855(SA6)[c], 1856(HY6)[c], 1857(ST7)[c], 1858(SU3)[c].

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867 (ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876 (WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

VOLUME 85 (1959)

JANUARY: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 1900(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 1909(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)[c], 1917(EM1)[c], 1918(HW1)[c], 1919(HY1)[c], 1920(PL1)[c], 1921(SA1)[c], 1922(ST1)[c], 1923(EM1), 1924(HW1), 1925(HW1), 1926 (PL1), 1927(HW), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO), 1957(CO1), 1958(CO1), 1959(CO1).

MARCH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968(SA2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977(WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 1986 (IR1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

APRIL: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998 (SM2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007 (HW2)[c], 2008(EM2)[c], 2009(ST4)[c], 2010(SM2)[c], 2011(SM2)[c], 2012(HY4)[c], 2013(PO2)[c].

MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)[c], 2041(PL2)[c], 2042(PP1)[c], 2043(ST5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PP1), 2047(PP1).

JUNE: 2048(CP1), 2049(CP1), 2050(CP1), 2051(CP1), 2052(CP1), 2053(CP1), 2054(CP1), 2055(CP1), 2056 (HY6), 2057(HY6), 2058(HY6), 2059(IR2), 2060(IR2), 2061(PO3), 2062(SM3), 2063(SM3), 2064(SM3), 2065 (ST6), 2066(WW2), 2067(WW2), 2068(WW2), 2069(WW2), 2070(WW2), 2071(WW2), 2072(CP1)[c], 2073(IR2)[c], 2074(PO3)[c], 2075(ST6)[c], 2076(HY6)[c], 2077(SM3)[c], 2078(WW2)[c].

JULY: 2079(HY7), 2080(HY7), 2081(HY7), 2082(HY7), 2083(HY7), 2084(HY7), 2085(HY7), 2086(SA4), 2087 (SA4), 2088(SA4), 2089(SA4), 2090(SA4), 2091(EM3), 2092(EM3), 2093(EM3), 2094(EM3), 2095(EM3), 2096 (EM3), 2097(HY7)[c], 2098(SA4)[c], 2099(EM3)[c], 2100(AT3), 2101(AT3), 2102(AT3), 2103(AT3), 2104(AT3), 2105(AT3), 2106(AT3), 2107(AT3), 2108(AT3), 2109(AT3), 2110(AT3), 2111(AT3), 2112(AT3), 2113(AT3), 2114(AT3), 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), 2120(AT3), 2121(AT3), 2122(AT3), 2123(AT3), 2124(AT3), 2125(AT3).

AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4), 2134 (SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)[c], 2139(PO4)[c], 2140(SM4)[c].

c. Discussion of several papers, grouped by divisions.

AMERICAN SOCIETY OF CIVIL ENGINEERS

OFFICERS FOR 1959

PRESIDENT

FRANCIS S. FRIEL

VICE-PRESIDENTS

Term expires October, 1959:
WALDO G. BOWMAN
SAMUEL B. MORRIS

Term expires October, 1960:
PAUL L. HOLLAND
LLOYD D. KNAPP

DIRECTORS

Term expires October, 1959:
CLINTON D. HANOVER, Jr.
E. LELAND DURKEE
HOWARD F. PECKWORTH
FINLEY B. LAVERTY
WILLIAM J. HEDLEY
RANDLE B. ALEXANDER

Term expires October, 1960:
PHILIP C. RUTLEDGE
WESTON S. EVANS
TILTON E. SHELBURNE
CRAIG P. HAZELET
DONALD H. MATTERN
JOHN E. RINNE

Term expires October, 1961:
THOMAS J. FRATAR
EARL F. O'BRIEN
DANIEL B. VENTRES
CHARLES W. BRITZIUS
WAYNE G. O'HARRA
FRED H. RHODES, JR.
N. T. VEATCH

PAST PRESIDENTS

Members of the Board

MASON G. LOCKWOOD
LOUIS R. HOWSON

EXECUTIVE SECRETARY

WILLIAM H. WISELY

ASSISTANT SECRETARY

E. LAWRENCE CHANDLER

ASSISTANT TREASURER

ENOCH R. NEEDLES

PROCEEDINGS OF THE SOCIETY

HAROLD T. LARSEN
Manager of Technical Publications

PAUL A. PARISI
Editor of Technical Publications

MARVIN SCHECHTER
Assistant Editor of Technical Publications

COMMITTEE ON PUBLICATIONS

HOWARD F. PECKWORTH, *Chairman*

PHILIP C. RUTLEDGE, *Vice-Chairman*

E. LELAND DURKEE
CHARLES W. BRITZIUS
TILTON E. SHELBURNE
FRED H. RHODES, JR.

PO 4

AUGUST 1959 — 29 NO. PO 4
VOLUME 85 PART 2

Your attention is invited

NEWS OF THE POWER DIVISION OF ASCE

JOURNAL OF THE POWER DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES

POWER DIVISION

Proceedings of the American Society of Civil Engineers

NEWS

August, 1959

Forthcoming Meetings

Arrangements are well advanced for the Power Division sessions at the Annual Meeting in Washington, D. C. The tentative program, which is subject to approval by Headquarters, comprises 16 papers:

First Session—Thursday, October 22, 8:30 A.M.

1st "Engineering and Operating Problems Peculiar to Navy Industrial Power Plants." L. F. Deming, Head of Power Generating Section, Bureau of Yards and Docks, Department of the Navy.

2nd "Pumped Storage Opportunities in Hydro-Electric Engineering." Adolph J. Ackerman, M. ASCE, Consulting Hydro-Electric Engineer.

3rd "More Power to Puerto Rico." Edwin T. Morris, Vice President, Raymond International Inc.

4th "An Engineer-Constructor's Role in World-Wide Power Expansion." John P. Buehler, M. ASCE, Manager of Hydro-Electric Power—Bechtel Corporation.

Second Session—Thursday, October 22, 2:30 P.M.

1st "Exporting American Engineering Experience." A. T. Larned, M. ASCE, Associate Consulting Engineer, Ebasco Services, Incorporated.

2nd "Technical Assistance to Foreign Countries by Bureau of Reclamation." Alfred R. Golze, M. ASCE, Assistant Commissioner Bureau of Reclamation.

3rd "Design of the Karadj Dam Project." Richard D. Harza, A. M. ASCE, Vice President, Harza Engineering Company. Robert E. Edbrooke, A. M. ASCE, Assistant Project Manager, Harza Engineering Company.

4th "Hydro Electric Opportunities and Influence of Load Growth." Stanford P. McCasland, M. ASCE, Chief Hydro-Electric Engineer, Ebasco International.

Note: No. 1959-29 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, PO 4, August, 1959.

Copyright 1959 by the American Society of Civil Engineers.

Third Session—Friday, October 23, 8:30 A.M.

1st "Some Unusual Civil Engineering Features of Thermal Electric Plants Built Overseas". Alexis L. Gluckman, M. ASCE, Supervising Structural Engineer, Gibbs & Hill, Inc.

2nd "Rehabilitation of Hwachon Dam and Power Plant." R. E. Hollich, M. ASCE, Head of Structural Section, International Engineering Company, Inc.

3rd "Sariya-Hydro-Electric Project." A. E. Eckberg, M. ASCE, Project Engineer, Charles T. Main, Inc.

4th "Thermal Electric Stations in Japan." O. D. Ides, Supervising Conc-Hyd Engineer, Ebasco Services, Incorporated. R. T. Richards, A. M. ASCE, Civil Engineer, Ebasco Services, Incorporated.

Fourth Session—Friday, October 23, 2:30 P.M.

1st "Design of Power Plants for Naval Shore Establishment." W. S. Foresman, Head Mechanical Engineering Section, Bureau of Yards and Docks, Department of the Navy.

2nd "The Design of the Furnas Hydro-Electric Project." J. W. Libby, A. M. ASCE, Chief Design Engineer, International Engineering Company, Inc.

3rd "Piratininga Station of San Paulo Light." H. M. Estes, Senior Structural Engineer, Stone & Webster Engineering Corporation. O. L. Hooper, M. ASCE, Chief Hydraulic Engineer, Stone & Webster Engineering Corporation.

4th "Civil Engineering Features of the Regla Steam Electric Station." George T. Ingalls, M. ASCE, Chief Structural Engineer, Ebasco International.

At the New Orleans convention March 7-11, 1960, a program of papers of local and general interest is being prepared. Engineers who desire to participate in this program are invited to communicate as soon as possible with Mr. Richard R. Randolph, Jr., Southern Services Incorporated, 600 North 18th St., Birmingham, Alabama.

The program for the Reno convention June 20-24, 1960, will be sponsored by Mr. John F. Bonner, Chairman of the Executive Committee, 245 Market St., San Francisco and the Local Program Chairman is Mr. George Thon, Manager of Hydraulic Engineering, Bechtel Corp., 101 California St., San Francisco, Calif. Our Western friends can always be depended on for an excellent program and engineers interested in contributing papers should communicate with either of these gentlemen.

Committee Activities

Control of Cracking in Masonry Dams.—Five papers prepared or sponsored by this committee were presented at the Los Angeles meeting in February 1959 and form a valuable contribution to this important subject.

Progress in Power Plant Design.—This Committee met in Cleveland May 6 and has appointed task forces to work on the following subjects:

Fish Passage Facilities at Power Plants.—Mr. G. H. Von Gunten, 1509 Melrose St., Walla Walla, Washington is sponsoring preparation of a bibliography on this subject, and papers will be prepared from time to time.

Thermal Power Plants.—Mr. M. G. Salzman, Ebasco Services, 2 Rector St., N. Y. and Mr. G. P. Palo, c/o TVA, Knoxville, Tenn. are sponsoring a selected bibliography and review of available information on civil engineering features of thermal plants.

Symposium on Rock Fill Dams.—Mr. J. B. Cooke, c/o P. G. & E. Co., 245 Market St., San Francisco, Calif., is sponsoring the preparation of comprehensive discussion on the many papers included in the symposium on Rock Fill Dams (Proceedings Symposium Series No. 3) and it is hoped that discussions and author's closures may be published as a companion volume to that symposium.

Economic Analysis of Size and Number of Generating Units at Hydroelectric Plants.—Mr. Von Gunten is arranging for presentation at a future meeting of papers on this subject by engineers of the U. S. Bureau of Reclamation and the U. S. Corps of Engineers.

Pumped Storage.—Mr. O. L. Hooper, c/o Stone & Webster Corp., 49 Federal St., Boston, Mass., and Mr. M. G. Salzman are continuing an investigation on this subject.

Engineers who can contribute worthwhile information on any of these subjects are cordially invited to communicate with the appropriate committee member.

Nuclear Energy.—Mr. A. G. Villepique has felt compelled to resign the Chairmanship of the Committee because of absence abroad and Mr. R. N. Bergstrom, c/o Sargeant & Lundy, 140 South Dearborn St., Chicago, Ill., has been appointed Chairman in his stead. It has been recommended that the purpose of the Power Division's Committee on Nuclear Energy be altered to read: "To investigate, and disseminate information relating to the economics and civil engineering features of nuclear power plants, including radioactive waste disposal. To cooperate in the exchange of information with similar committees formed by other bodies."

As members of the Power Division have no doubt realized, there is widespread interest and accelerated activity in the development of nuclear energy, and this Committee will perform an important function in this development.

Operation and Maintenance of Hydroelectric Generating Stations.—A progress report on "Intakes, Racks and Booms" has been prepared and other subjects are under continued study. The Committee Chairman is Mr. K. O. Strange, c/o The Washington Water Power Co., Spokane, Washington.

Honors

Mr. J. George Thon, Manager of Hydraulic Engineering, Bechtel Corporation and Mr. Gordon L. Coltrin, Senior Civil Engineer, Pacific Gas and Electric Co. were awarded the Thomas Fitch Rowland Prize for their paper on the "Morro Bay Steam Electric Plant."

Mr. James P. Growdon, Consulting Engineer, was awarded the Rickey Medal for his papers:

"Rockfill Dams—Nantahala Sloping Core Dam"
"Rockfill Dams—Dams with Sloping Earth Cores"
"Rockfill Dams—Performance of Seven Sloping Core Dams."

and for his contribution over many years to the science and progress of hydroelectric engineering.

These prizes will be presented at the October Convention in Washington, D. C. Congratulations to these gentlemen on their well-merited recognition.

Early Transactions Volumes Obtainable

The feasibility of reproducing the first ten volumes of ASCE Transactions (1872-1881) has been studied. It has been decided that these historic volumes could be reproduced at a cost that would permit a top price of $150 for the ten-volume set. If more than 100 engineers, or libraries, indicate an interest in obtaining such a set, the project will be undertaken. If the endeavor is successful, other rare volumes of Transactions will be reprinted.

Engineers interested in obtaining the ten-volume set should write to the Executive Secretary of ASCE, 33 West 39th Street, New York 18, N. Y.

Robert A. Sutherland
Newsletter Editor